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PLACING TARRED FELT ALONG A LONGITUDINAL JOINT

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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THE STRUCTURAL DESIGN OF CONCRETE PAVEMENTS

BY THE DIVISION OF TESTS, BUREAU OF PUBLIC ROADS

Reported by L. W. TELLER, Senior Engineer of Tests, and EARL C. SUTHERLAND, Associate Highway Engineer

PART 4.—A STUDY OF THE STRUCTURAL ACTION OF SEVERAL TYPES OF TRANSVERSE AND LONGITUDINAL JOINT DESIGNS [1]

PRECEDING REPORTS on parts of this investigation have presented: (1) A general description of the entire project and of the methods employed in making the tests (part 1); (2) a discussion of the effects of temperature and moisture variations on the size, shape, and load-carrying ability of pavement slabs as observed during the course of these studies (part 2); and (3) a discussion of the results of tests on various pavement cross-sections (part 3).

This report contains a description of the studies that were made of the structural action of the several transverse and longitudinal joints included in the investigation. In presenting this material, certain descriptive matter will be repeated from the preceding reports for the purpose of amplification together with such data from parts 2 and 3 as are necessary for an adequate treatment of the subject.

In dealing with the subject of the design and use of joints in concrete pavements, it is of considerable interest to look backward over the period of concrete pavement construction and trace the development of theory and practice in regard to joint construction. This development will be sketched rather briefly.

A number of concrete pavements were built in Europe and in the United States long before the beginning of the present century. There is mention of one constructed in Inverness, Scotland, as early as 1865,[2] while in this country one of the earliest of which there is an authentic record is that constructed in Bellefontaine, Ohio, in 1892. So little information is given in the accounts of these early concrete pavements that in most cases no details of the spacing and design of the joints are available. It appears, however, that the joints were simply small spaces left between adjacent slabs and were intended to be filled with earth, although as far back as 1871 a patent was granted that gave the inventor rights covering the use of gum, tar, rubber, or other water-repellent substances as a filler for joints in pavements made of concrete blocks.[3] Some mention is made in engineering literature of the use of pitch and of creosote oil for the same purpose at about the same time.

EARLY JOINTS DESIGNED TO PROTECT SLAB EDGES FROM DAMAGE BY STEEL-TIRED WHEELS

The Bellefontaine pavement was laid in small slabs or blocks 5 or 6 feet square and tarred paper was placed between the blocks to allow for expansion.[4] It is interesting to note that with this small-slab construction practically no cracking has occurred in this pavement during more than 40 years of service. At the time it was constructed all traffic was carried on steel-tired wheels and much damage was done to the edges of the slabs by teamsters who drove so that their wagon wheels followed the joints.[5] Efforts to protect slab edges from the damaging action of steel-tired wheels seem to have been the dominant throught in the early consideration of joint design.

Figure 1 shows a drawing of what is one of the first, if not the first, joint designs for concrete pavements patented in this country.[6] The object of this design, as stated in the patent, was to allow adjacent blocks to heave without injury to their edges. Direct expansion apparently was not a consideration. It was specified that the metal forming the joint should be stiff enough to permit tamping the concrete around it, yet light enough to crush in the event that heaving occurred.

The decade between 1900 and 1910 might well be considered as the early formative period in concrete pavement history. A new type of pavement was developing and the literature of this period contains many inquiries as to how concrete roads should be built, with little or no information available to supply the answers. The joint, intended to provide for expansion and to control cracking, made its appearance, although it continued to be a simple opening between slabs. Breaking of the slab edges under the action of the steel-tired wheels was still a serious problem and its effect is reflected in the staggered and oblique joint designs of the period. Some pavements of which there is a record of the joint construction are described briefly as follows:

Grand Rapids, Mich., 1901–02: In a two-course pavement, joints were placed along the curbs and transversely at intervals of 25 feet in the base course. The width of these joints is not recorded. The top course was laid in alternate blocks 6 feet square with expansion joints one-fourth inch in width between blocks. These joints were filled with asphalt.

Toronto, Canada, 1902: This pavement was laid in blocks 20 feet square with ¾-inch expansion joints between the blocks. The joints were filled with "paving pitch." It is reported that under heavy traffic the edges of the slabs shattered badly.

Richmond, Ind., 1903–04: The earliest concrete pavements in this city date from 1896. Of the early pavements no information about the joint design was found, although it appears that small slabs were used. The pavements laid in 1903–4 were in large slabs with expansion joints 1 inch wide. It was reported that these wide joints were troublesome because of chipping at the slab edges. Mention of temperature cracking in connection with this pavement appeared in the early reports.

[1] A series of five articles has been planned. Parts 1, 2, and 3 have appeared in PUBLIC ROADS, vol. 16, nos. 8, 9, and 10, October, November, and December 1935, respectively. Because of its length, Part 4 will be presented in two issues of PUBLIC ROADS. The second installment will appear in the October issue.

[2] Cement and Concrete—A general refrence book, 1929. Portland Cement Association, p. 49.

[3] United States Patent no. 120268 granted Oct. 24, 1871, to H. A. Gunther.

[4] Portland Cement Pavement, by G. W. Bartholomew, Jr., Engineering News, vol. 33, no. 1, Jan. 3, 1895, p. 5.

[5] The Concrete Pavements of Bellefontaine, Ohio, by Prof. F. H. Eno, Engineering News, vol. 51, no. 1, Jan. 7, 1904, p. 15.

[6] United States Patent no. 312897 granted Feb. 24, 1885, to C. F. Rapp.

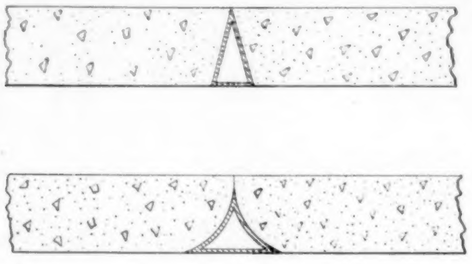


FIGURE 1.—ONE OF THE FIRST JOINT DESIGNS FOR CONCRETE PAVEMENTS PATENTED IN THE UNITED STATES.

Washington, D. C., 1906: This pavement was laid in slabs 100 feet long separated by 1-inch joints filled with a bituminous material.

City of Panama, 1906–7: It is recorded that the first concrete paving in this city consisted of slabs 10 feet in length. On wide streets the pavement was divided longitudinally at the center and the slabs were staggered on either side of this joint.

Because of difficulties with chipping and spalling along the joints, commercial companies specializing in concrete pavement construction began gradually to increase the spacing between joints. This practice continued for many years and culminated in the construction of hundreds of miles of concrete pavements in which the only joints constructed were at places where the paving operation was stopped for some reason.

That some engineers at this time appreciated the advantages of crack control in concrete pavements is evidenced by the following quotation from a patent for a joint design granted to Mr. R. Kieserling, a German citizen, by the United States Patent Office in 1906: [7]

"As it is well known, irregularly running cracks appear after a short time in paving made from concrete or other cement mixtures, which lead to the destruction of the pavement. * * * I avoid this irregular formation of cracks by providing for the occurrence of cracks at definite places and causing them to run in a direction previously determined upon." Mr. Kieserling's design for accomplishing this control of cracking is shown in figure 2.

WIDE DIFFERENCES FOUND IN STRUCTURAL DETAILS OF EARLY JOINTS

By 1910 the automobile had demonstrated itself to be a practical machine and with its increasing use came the demand for more and better highways, particularly interstate highways. The stimulus thus given to road building is reflected in the increased discussion of concrete pavement design. Spalling and chipping at the joints and separation of adjacent slabs, both vertically and horizontally, were troubles which gave a great deal of concern in these early pavements. To overcome chipping, diagonal joints and edges armored with metal were frequently recommended. For the displacement at the joints, particularly the longitudinal joints, numerous suggestions appeared—more attention to drainage, reinforcement to prevent longitudinal cracks, dished subgrade to provide a thicker pavement at the center of the road, and rolled stone subbases—each had its advocates.

[7] United States Patent No. 839600, granted Dec. 25, 1906, to R. Kieserling, Germany.

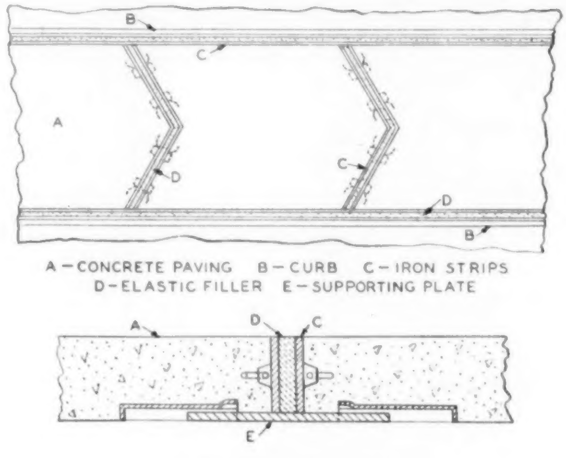


FIGURE 2.—AN EARLY PATENTED JOINT DESIGN FOR CONCRETE PAVEMENTS.

Speaking editorially, one of the leading engineering journals of the country said of the concrete pavement joint designs of this period: [8]

> Practice exhibits a heterogeneous array of expansion joint details, spacings, and arrangements. This is most true of transverse joint practice. The plan is general of placing joints between pavement edge and curb and, when railway tracks occupy the streets, of placing joints on each side of the tracks just outside the tie ends. There is no similar uniformity in transverse expansion joint practice. They are spaced 25, 30, 37½, 50, 60, and 100 feet apart, and the most common spacings are perhaps 25 and 30 feet. Usually they are square across the roadway but various diagonal arrangements are employed. Structurally the differences are wide. Joints with metal guard plates, joints with rounded edges only, joints of all widths from ¼ to 1 inch, joints with fillers of a dozen characters are employed.

As mentioned previously, some of the State highway departments adopted the practice of laying concrete pavements without joints except at points where the construction operation was stopped. By 1915 a number of States were building their pavements in this manner. The reasons prompting this policy were described by one State highway engineer,[9] who stated that the occurrence of transverse cracks had been almost as erratic in pavements with the joints spaced 50 feet apart as in those in which the spacing was 100 feet. The difficulty of constructing smooth surfaces in the vicinity of the joints and the chipping of the slab edges at the joints under the action of traffic were also important considerations.

At the tenth annual convention of the American Concrete Institute (1914) certain recommended specifications for concrete pavement construction were adopted, and the recommendations relative to joint construction probably reflect the thought as to the best practice at that time.

In these specifications it was recommended that transverse joints should be not less than one-fourth nor more than three-eighths inch in width and should be placed across the pavement perpendicular to the center line, not more than 35 feet apart. It was further recommended that a longitudinal joint not less than one-

[8] Engineering and Contracting, vol. 40, no. 2, July 9, 1913.

[9] H. E. Bilger, State road engineer of Illinois, in a paper delivered before the Illinois Society of Engineers and Surveyors 1915. Also Engineering and Contracting, vol. 43, no. 11, Mar. 17, 1915, pp. 254–5.

fourth inch in width should be constructed between the curb and the pavement and that all joints should extend completely through the pavement and be perpendicular to its surface. Also, the concrete at transverse joints should be protected with soft-steel, joint-protection plates rigidly attached to the concrete, the surface edges of the metal plates to conform to the surfaces of the concrete. All joints found to be more than one-fourth inch too high or one-half inch too low were to be removed. It was specified further that all joints were to be formed by inserting, during construction, and leaving in place the required thickness of joint filler, this filler to extend through the entire thickness of the pavement.

IN 1917 LOAD TRANSFER APPEARED AS A FACTOR IN JOINT DESIGN

It will be observed that provision for expansion and protection of the joint edges are dominant considerations and that mutual support through transfer of load is not mentioned as a joint requirement. The smoothness tolerances are of interest in contrast with the specifications of today.

Load transfer as a factor in joint design was soon to appear, however. In the design of a concrete pavement constructed between two Army camps near Newport News, Va., during the winter of 1917–18, steel dowels were placed across all transverse joints for the stated purpose of transmitting load across the joint by shear.[10] The joints were three-eighths inch in width and four three-quarter inch diameter steel dowels were used in the 20-foot pavement width. It was recommended that eight rather than four dowels be used. Heavy truck traffic during the World War period apparently failed to damage these joints.

Following the World War the use of steel dowels spread rapidly wherever concrete pavement was being laid and has continued up to the present time.

Although the principle had been well known for many years, one of the earliest references to the use of the weakened-plane contraction joint for crack control appears in connection with a pavement laid in West Virginia in 1919.[11] It was constructed by grooving the bottom of the slab by setting a thin board on edge on the subgrade, the width of the board being approximately one-half the thickness of the pavement. The concrete was then cast over the board.

Soon after this the recommendation appeared that this type of joint be formed by a board one-fourth inch thick and 6 inches wide so cupped or warped as to give a tongue-and-groove effect to adjoining slabs, thus preventing uneven settlement of the abutting edges.

Figure 3 shows typical joint designs for which patents were granted in this country during the decade following 1910. The essential feature of the design shown in figure 3–A was the use of steel protection plates at the joint edges, tied in to a general system of reinforcement. The object of the design shown in figure 3–B was to permit the placing of the joint filler in advance of the concreting operation. It will be noted that an air chamber was provided to take care of the filler material during expansion. Figure 3–C shows a design intended to protect the edges of the slabs and at the same time serve as a container for the filler.

The use of a steel T-section embedded in the plastic filler material was proposed in the design shown in figure 3–D, the T-section presumably serving to protect the edges of the concrete. While the design shown in figure 3–E was apparently intended primarily to provide a sliding key or bridge in order to hold the filler material, both the design and the claim contain the germ of an idea that appears in many of the joint designs being promoted today. Figure 3–F shows a heavily armored expansion joint in some respects quite similar to designs recently proposed although the idea of load transfer does not appear in the claims.

The design shown in figure 3–G is definitely intended to provide "an interlocking engagement of the adjacent concrete sections" although the compressible material which is interposed between the corrugated plates, together with separation caused by contraction, would probably completely defeat the purpose. Figure 3–H shows a design that includes the use of dowels which are not bonded to the concrete, and are installed for the stated purpose of maintaining the engaged sections of concrete in the proper relation to each other and at the same time permitting independent expansion and contraction.

NEED FOR BETTER EXPANSION AND CONTRACTION JOINTS RECOGNIZED

The disappearance of steel-tired vehicles from the highway, a change which accelerated rapidly during the period following the World War, eliminated what had been one of the worst problems in joint design, i. e., chipping of the slab edges. The result was the general omission of the steel, edge-protection plates from joint designs. A new trouble appeared, however, with the increased use of concrete pavements. Expansion failures known as "blow-ups" began to appear 3 or 4 years after the laying of the pavement, and the seriousness of some of these created a renewed interest in joints providing relief for expansion.

The desire to improve the appearance of concrete pavements by control of cracking led to the more widespread use of the so-called "contraction joints." As already noted, the earliest joints of this type were constructed by grooving the bottom of the slab. The irregularity of the crack on the slab surface, coupled with the difficulty in sealing these joints effectively, led to an unfavorable reaction which resulted in the general abandonment of this design. Shortly after 1920 a weakened-plane joint appeared in which the upper surface of the slab was grooved. While more difficult to construct, it obviated the difficulties just noted and, with the development of mechanical methods for grooving the concrete at the time of construction, this type of contraction joint came into rather widespread use.

The decade following 1920 also saw the general adoption of longitudinal joints that divide the pavement into slabs approximately 10 feet wide. Experience showed that such joints practically eliminated longitudinal cracking and, since this width is about what is required for a single lane of traffic, the practice of building pavements in slabs about 10 feet wide has developed naturally and has resulted in effective control of longitudinal cracking.

During the early part of this decade researches such as the test road at Pittsburg, Calif., the Bates road tests in Illinois, and experiments of the Bureau of Public Roads at Arlington, Va., developed certain basic facts concerning the effect of loads on pavement slabs of various designs. In all of these researches the need for strengthening slab edges was definitely indicated. Free edges of slabs can be strengthened most simply by increasing the slab depth, but where the slab adjoins others the possibility for inter-slab support as a means

[10] Engineering News-Record, vol. 88, no. 9, Mar. 2, 1922, pp. 357–8.
[11] Engineering News-Record, vol. 85, no. 7, Aug. 12, 1920, p. 305.

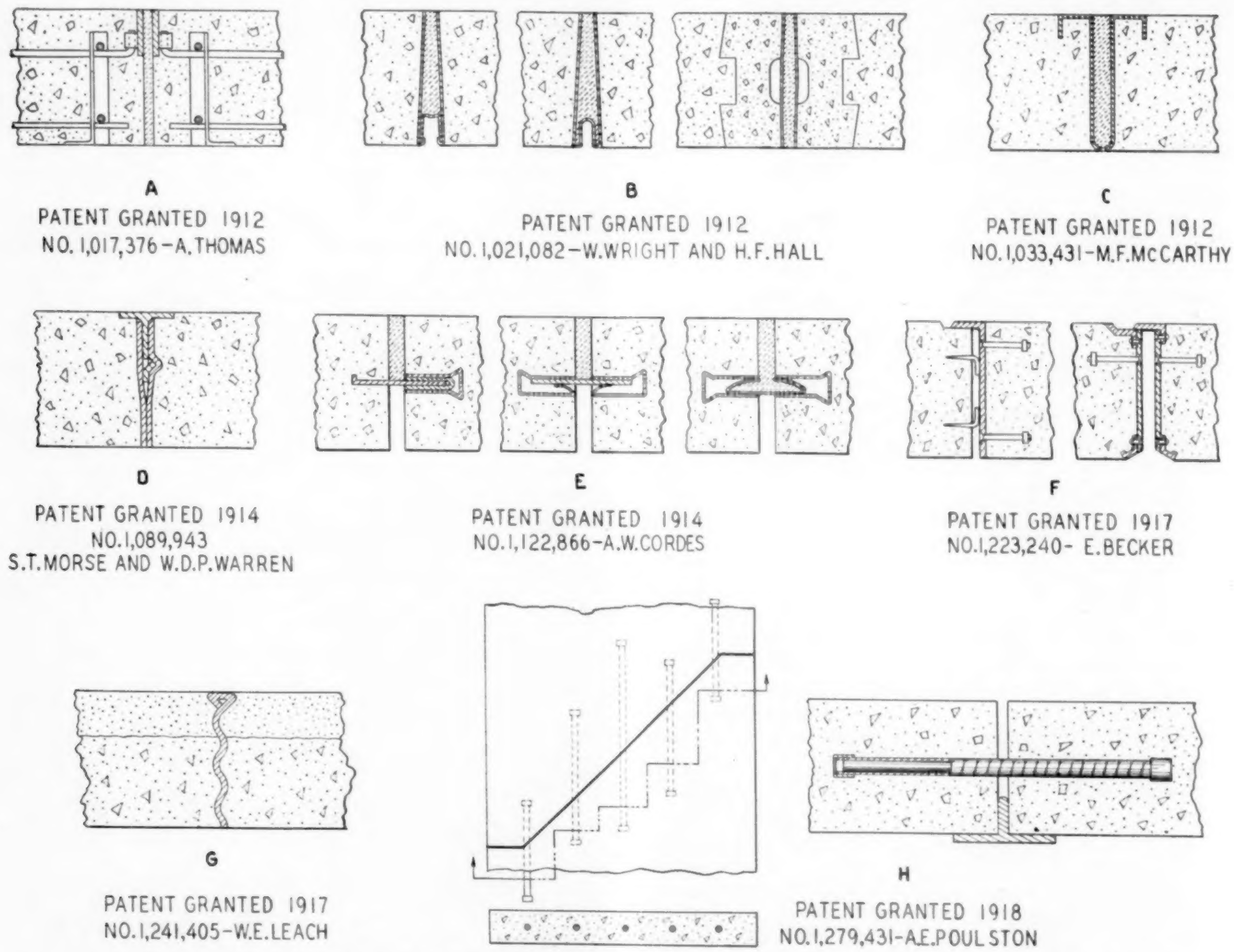


Figure 3.—Some Joint Designs That Were Patented in the United States Between 1910 and 1919.

for strengthening the edges has long been recognized and has led to many proposals for joint designs in which varying degrees of interlocking action are developed. The use of transverse joint designs in which some form of load-transfer mechanism is incorporated has become quite general, the round, steel dowel bar being the most common.

Efforts were also made to strengthen structurally, by systems of steel reinforcement, certain parts of the slab, usually the edges and corners. Some of these proposed systems were very simple; others were quite extensive and complicated.

In figure 4 are shown a number of typical joint designs for which patents were granted between 1919 and 1929. It is of interest to compare this group with that shown in figure 3 and note how the changes in ideas about joint design that have just been discussed are reflected in these two groups of designs. The idea of edge protection disappears and the idea of load transfer appears as the most important factor in the design.

Figure 4–A shows a method for the control of cracking by means of a transverse, steel parting strip so deformed as to create corrugations of various shapes to provide for an interlocking action of the two slab edges. Figure 4–B is similar except that complete separation is provided without cracking and a single approximately rectangular tongue and groove is formed. Figure 4–C shows a deformed metal plate intended for longitudinal joints and forming a triangular tongue and groove similar to that used in one of the test sections. Figure 4–D shows a doweled joint with a short cap to provide for end freedom of the dowel during expansion. In figure 4–E are shown several designs incorporating various methods of load transfer together with a collapsible metal box intended to form the opening between the slabs at the time of construction and to remain in place as a seal afterward.

The use on one slab of rounded projections that engage sockets of the same shape on the adjoining slab is proposed for load transfer in the longitudinal joint shown in figure 4–F. The use of bonded dowels is contemplated in this design. Figure 4–G shows an expansion joint in which inter-slab action is obtained by both a concrete tongue and groove and by steel bars which pass from slab to slab.

CLOSER SPACING OF TRANSVERSE JOINTS GRADUALLY ADOPTED

The great differences of opinion as to how far apart joints should be placed, which were remarked in the 1915 editorial, persisted for many years. In 1931 three States used expansion joints only at bridge approaches while several others employed them only under special conditions (which frequently meant only at bridge approaches). The remainder, with the exception of one State, installed expansion joints at intervals of

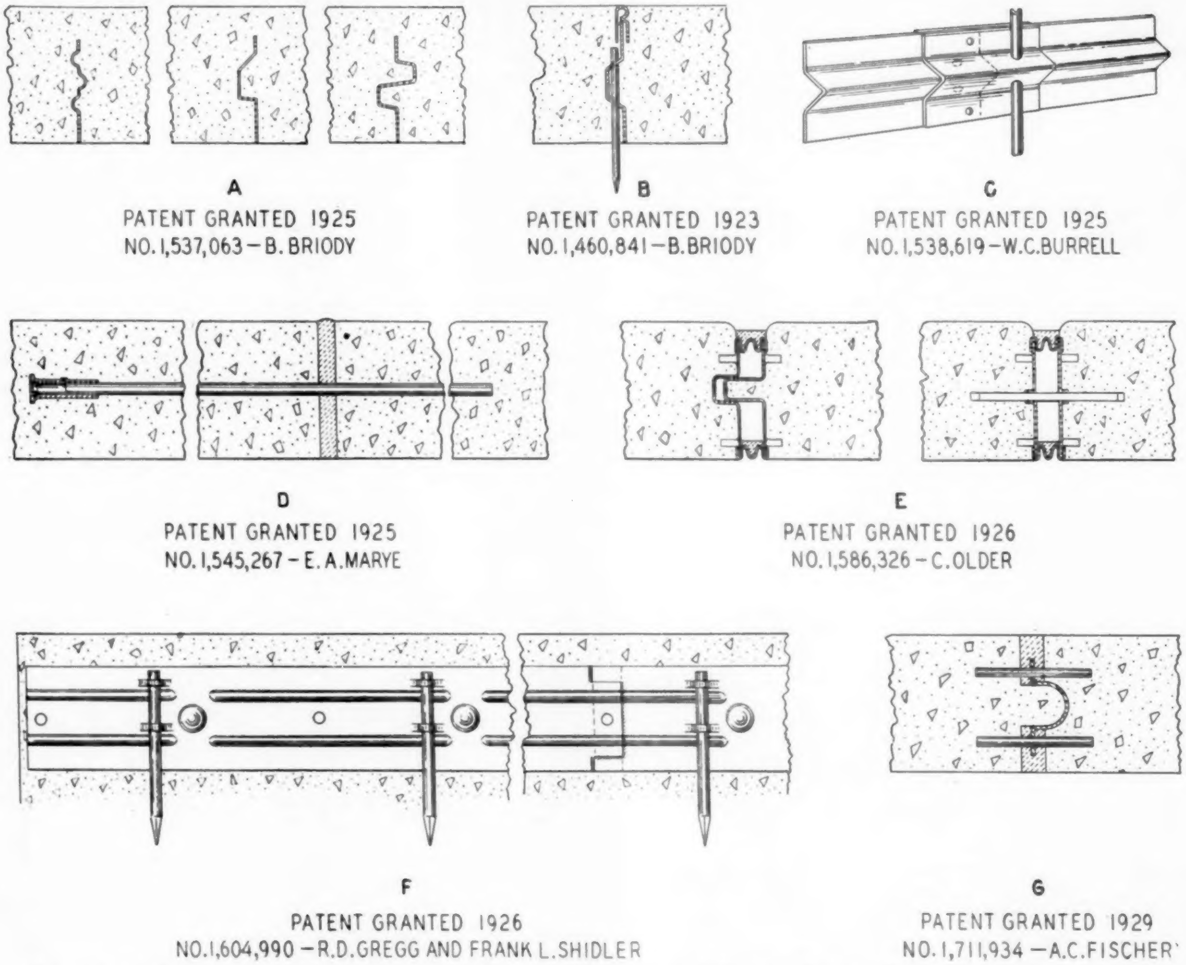


FIGURE 4.—SOME JOINT DESIGNS THAT WERE PATENTED IN THE UNITED STATES BETWEEN 1919 AND 1929.

approximately 100 feet or less. This one State constructed a 4-inch expansion joint at approximately 800-foot intervals and used no other transverse joints in concrete pavements.

By 1934 all of the States, with but one exception, were installing expansion joints at intervals of 100 feet or less (and in this one the interval used was 150 feet). Also, the adoption by many States of the policy of using contraction joints between the expansion joints resulted in a still further reduction of the interval between transverse joints. During the early part of 1934 the Bureau of Public Roads made the requirement that on Federal-aid road construction expansion joints should be provided at intervals of not more than 100 feet and that in plain-concrete slabs transverse joints should be placed at intervals not exceeding 30 feet. It was required also that the width of expansion joints should be not less than three-fourths nor more than 1 inch and that some provision for load transfer should be made in all transverse joint installations. These requirements for Federal-aid construction have probably accelerated the trend toward a shorter distance between joints, a trend that has been discernible for a number of years in spite of the wide variation of opinion which has existed.

Although there is widespread acceptance of the desirability of inter-slab load support at transverse joints, there is both a wide divergence of opinion as to how it should be accomplished and a decided lack of agreement on the fundamental structural requirements of a satisfactory joint design. This condition is caused principally by a lack of conclusive evidence from tests or other sources as to what these requirements should be.

In 1927 Westergaard published an analytical treatment of the action of a doweled joint under load.[12] This valuable contribution to the general subject of joint design has apparently not been given the attention it deserves. The analysis showed the effect of dowel spacing on the stresses directly under a load acting at a doweled edge of a pavement slab, throwing new light on the critical stresses in the vicinity of joints of this type. It is indicated that dowels, even under the ideal conditions that were assumed, must be placed very close together if they are to be reasonably effective as a means for transferring load.

Aside from Westergaard's analysis there has been little information available except for occasional reports of the observed service behavior of certain joint installations.

An examination of the designs shown in figure 5 will reveal how widely opinions vary as to what is required structurally in joint action. It will be noted that some believe that a joint should be shear resistant

[12] Analysis of Stresses in Concrete Roads Caused by Variations of Temperature. PUBLIC ROADS, vol. 8, no. 3, May 1927.

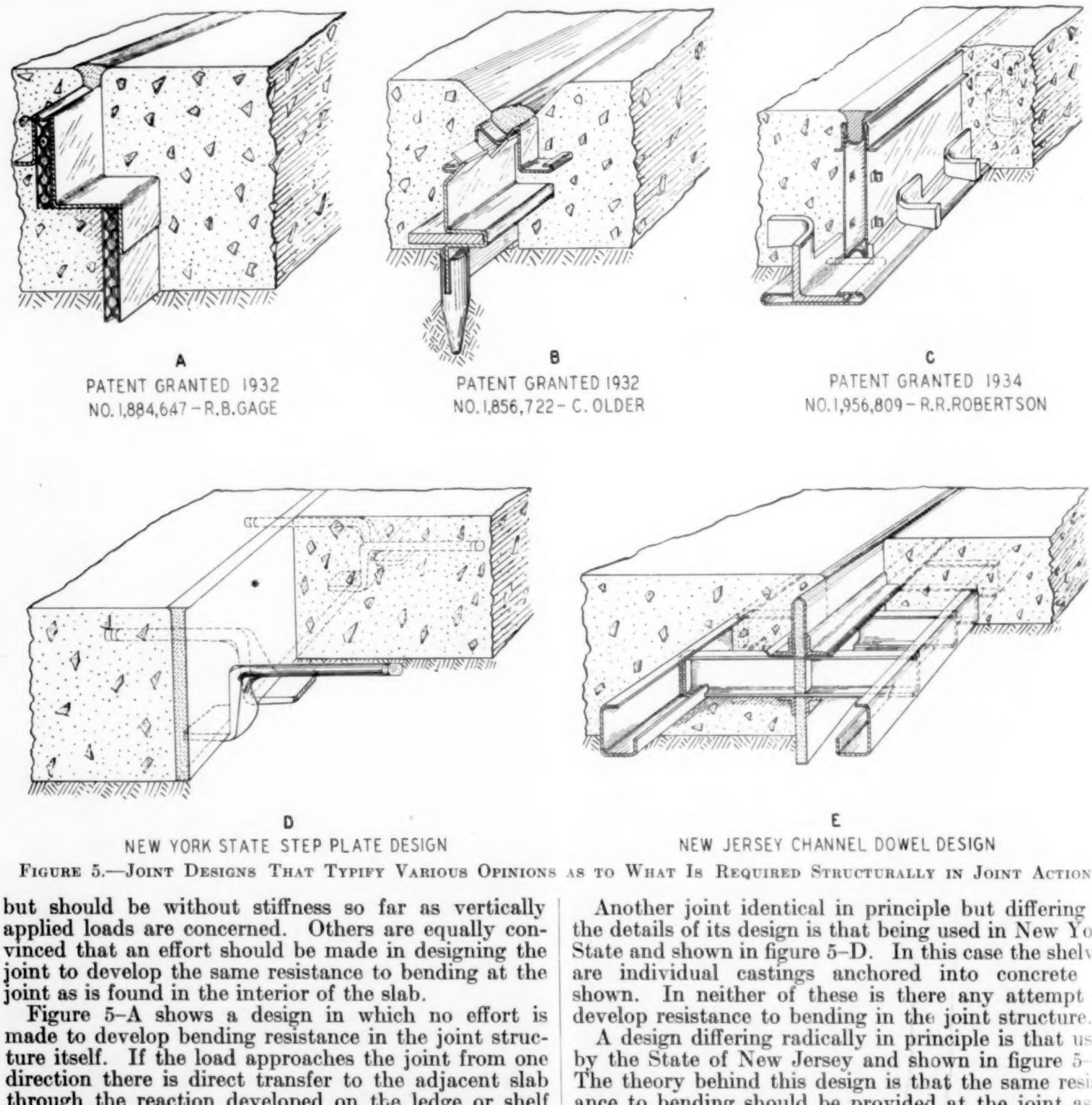


FIGURE 5.—JOINT DESIGNS THAT TYPIFY VARIOUS OPINIONS AS TO WHAT IS REQUIRED STRUCTURALLY IN JOINT ACTION.

but should be without stiffness so far as vertically applied loads are concerned. Others are equally convinced that an effort should be made in designing the joint to develop the same resistance to bending at the joint as is found in the interior of the slab.

Figure 5–A shows a design in which no effort is made to develop bending resistance in the joint structure itself. If the load approaches the joint from one direction there is direct transfer to the adjacent slab through the reaction developed on the ledge or shelf on the adjacent slab. The joint in this case acts somewhat as a free hinge. If the load approaches from the opposite direction there can be no transfer of load.

One of the designs, shown in figure 5–B, shows a steel plate running the length of the joint and fitting into grooves or recesses formed into the two opposing slab ends. The plate acts as a key or spline and by its stiffness transfers part of the load across the joint. The flexibility of the plate permits a certain amount of hinge action to occur. Figure 5–C shows another design in which one slab rests on a shelf on a slab opposite. The shelf or ledge in this case is of steel and is anchored into the concrete of the slab end. In order to obtain the same support for each slab, the shelf angles are cut into short sections, half of the projections extending from each slab and so staggered that they intermesh, giving a typical hinge construction.

Another joint identical in principle but differing in the details of its design is that being used in New York State and shown in figure 5–D. In this case the shelves are individual castings anchored into concrete as shown. In neither of these is there any attempt to develop resistance to bending in the joint structure.

A design differing radically in principle is that used by the State of New Jersey and shown in figure 5–E. The theory behind this design is that the same resistance to bending should be provided at the joint as is found at the other points along the slab, and the series of stiff members which span the joint in this design are for this purpose.

JOINTS MAY ACT TO RELIEVE STRESSES RESULTING FROM EXPANSION, CONTRACTION, OR WARPING

A feature of joint design that has given considerable concern and that has been and is still being given a great deal of study is the filling and sealing of expansion joints. It presents a related but separate problem and was not a part of the investigation that is being reported in this series of papers.

In this brief review it has been noted that joints appeared with the first use of concrete for paving, probably the division of the early pavements into small units being as much for convenience in construction as for any other reason. Later, expansion joints as such

appeared with the expressed idea that their use would control the cracking which inevitably occurred. Difficulty in the construction of joints and their apparent ineffectiveness as a means of crack control led to a reaction that resulted in a decreased use of joints for a period. The results obtained by this policy were not altogether satisfactory, however, and the continued urge for smoother and better appearing pavements led to the introduction of what have been called contraction joints placed between expansion joints, the length of the slab units being gradually decreased. Load transfer as a recognized factor in joint design appeared after the World War and is now quite generally considered to be an essential requirement.

The importance of freely acting joints as a means for the relief of stresses developed by restrained temperature (and possibly moisture) warping is as yet not generally appreciated, although the results obtained with the longitudinal center joint have been evident for years and both the theoretical and experimental indications of the importance of warping stresses were pointed out before the Highway Research Board nearly a decade ago.[13] [14]

As shown in the preceding papers of this series, the present investigation has developed a considerable amount of information about the magnitude and the distribution of warping stresses. This information, much of which is new, emphasizes the necessity for controlling these stresses in concrete pavements if adequate wheel-load resistance is to be provided. The data that have been presented relative to warping stresses bear directly, therefore, on one important function that a joint should be designed to perform.

Thus it appears that joints in concrete pavements may be classified according to their intended function, as follows:

1. Those designed to provide space in which unrestrained expansion can occur.

2. Those designed for the relief or control of the direct tensile stresses caused by restrained contraction.

3. Those designed to permit warping to occur, thus reducing restraint and controlling the magnitude of the bending stresses developed by restrained warping.

Obviously a joint may and frequently does perform all three of these functions. An expansion joint, for example, may permit unrestrained expansion, contraction, and warping, while a joint of the so-called contraction type may actually benefit the pavement more by its ability to relieve warping stresses than by its intended function of relieving direct tensile stresses caused by contraction.

It should be kept in mind that joints are needed in concrete pavements for the one purpose of reducing as much as possible the stresses resulting from causes other than applied loads in order that the natural stress resistance of the pavement may be conserved to the greatest possible extent for carrying the loads of traffic.

A joint is potentially a point of structural weakness and may limit the load-carrying capacity of the entire pavement so that it is important to examine joint designs from this standpoint as well as for their ability to permit unrestrained expansion, contraction, and warping.

[13] Analysis of Stresses in Concrete Pavements Due to Variations of Temperature, by H. M. Westergaard, Proceedings, Sixth Annual Meeting, Highway Research Board, December 1926, pp. 201-215.

[14] Progress Report on the Experimental Curing Slabs at Arlington, Virginia, by J. T. Pauls, Proceedings, Sixth Annual Meeting, Highway Research Board, December 1926, pp. 192-201.

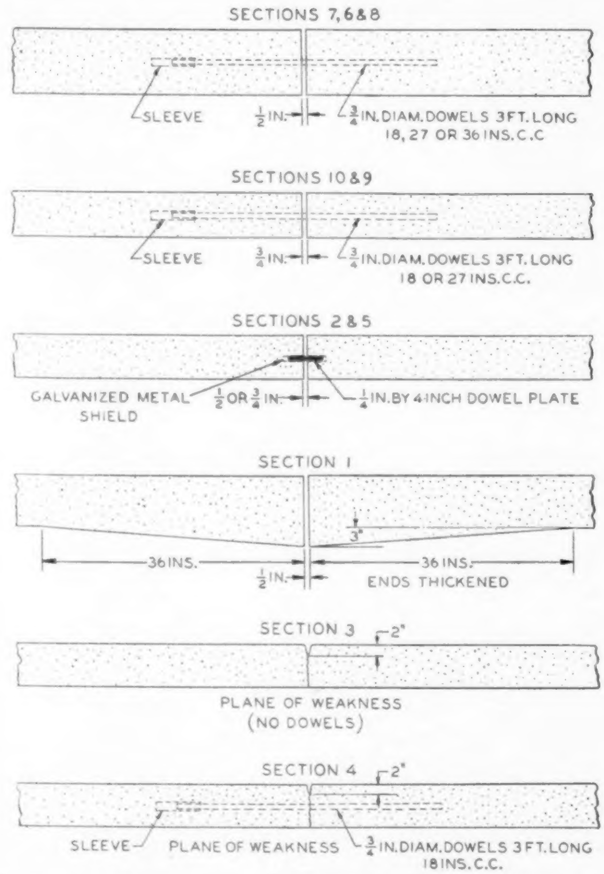


FIGURE 6.—DESIGNS OF TRANSVERSE JOINTS INCLUDED IN THE INVESTIGATION. THE BOND WAS DESTROYED ON ALL DOWELS ACROSS TRANSVERSE JOINTS BY PAINTING AND GREASING.

In studying the structural action of joints in this investigation, each joint was subjected to tests to determine its relative effectiveness for:

1. Permitting unrestrained expansion and contraction.
2. Permitting unrestrained warping at the joint.
3. Reducing the structural weakness created by the break in the slab continuity at the joint.

INSTALLATION AND DETAILS OF TRANSVERSE JOINTS DESCRIBED

In the first paper of this series there was given a brief description of the 10 transverse and the 10 longitudinal joints that were included in the pavement sections built for this investigation. Before beginning the description of the tests and the discussion of the results, it is desirable to refer again to the details of these joints.

The details of the several types of transverse joints studied are shown in figure 6. The joints are all classed as expansion and contraction joints with the exception of the two transverse plane-of-weakness or dummy joints that were incorporated in sections 3 and 4. These two are primarily contraction joints. The transverse joints are divided into four groups, according to type.

The first group comprises the dowelled expansion and contraction joints found in sections 6, 7, 8, 9, and 10. In this group the dowels were round, rolled-steel bars three-fourths inch in diameter and 3 feet in length in all cases but both the spacing of the dowels and the distance between the abutting slab ends (or joint opening) were varied, as shown in table 1, in order to determine the

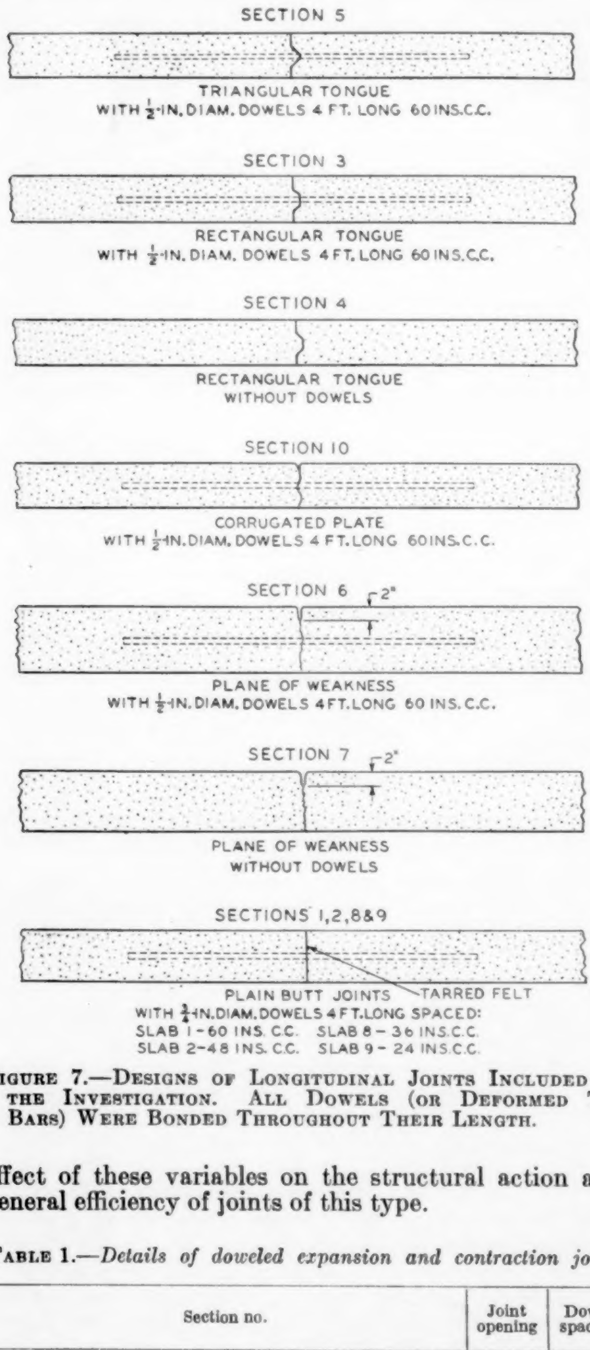


FIGURE 7.—DESIGNS OF LONGITUDINAL JOINTS INCLUDED IN THE INVESTIGATION. ALL DOWELS (OR DEFORMED TIE BARS) WERE BONDED THROUGHOUT THEIR LENGTH.

effect of these variables on the structural action and general efficiency of joints of this type.

TABLE 1.—*Details of doweled expansion and contraction joints*

Section no.	Joint opening	Dowel spacing
	Inch	*Inches*
6	½	27
7	½	18
8	½	36
9	¾	27
10	¾	18

At the time of installation the dowels were carefully painted and coated with grease to prevent bonding with the concrete, and special pains were taken to insure that all of the dowels were parallel to the subgrade and to the longitudinal axis of the pavement section. As will be noted in the drawings (fig. 6) a short cap or sleeve on one end of each dowel permits free longitudinal movement of the dowel within the concrete.

In the second group of transverse joints are the two plate-dowel designs found in sections 2 and 5, the only difference between the two being in the width of the joint opening. In each the steel dowel plate is one-fourth by 4 inches in section and is continuous for the full 10-foot width of the pavement slab. The bonding of the dowel plates to the concrete was prevented by an encasing shield of sheet metal which extends beyond the edges of the dowel plate in such a manner as to provide for free movement of the dowel plate during expansion and contraction of the pavement. The widths of the joint openings employed in these two joints are one-half inch (sec. 2) and three-fourths inch (sec. 5).

The third type of transverse joint is that in which the thickness of the slab ends abutting the joint has been increased above the thickness of the interior of the slab for the purpose of strengthening the transverse slab edges. In this design no load transfer is attempted since no inter-support is necessary; hence there is no direct connection between adjacent slabs. Such a joint was placed in section 1. The ends of this section at the open joints were also thickened.

The fourth and last type of transverse joint included in this investigation is the weakened-plane or dummy joint found in sections 3 and 4. These transverse joints were constructed in the same manner as the longitudinal joint of the same type except that bonding of the dowels was prevented in one (section 4), while in the other (section 3) all dowels were omitted. The spacing of the dowels in section 4 is 18 inches.

INSTALLATION AND DETAILS OF LONGITUDINAL JOINTS DESCRIBED

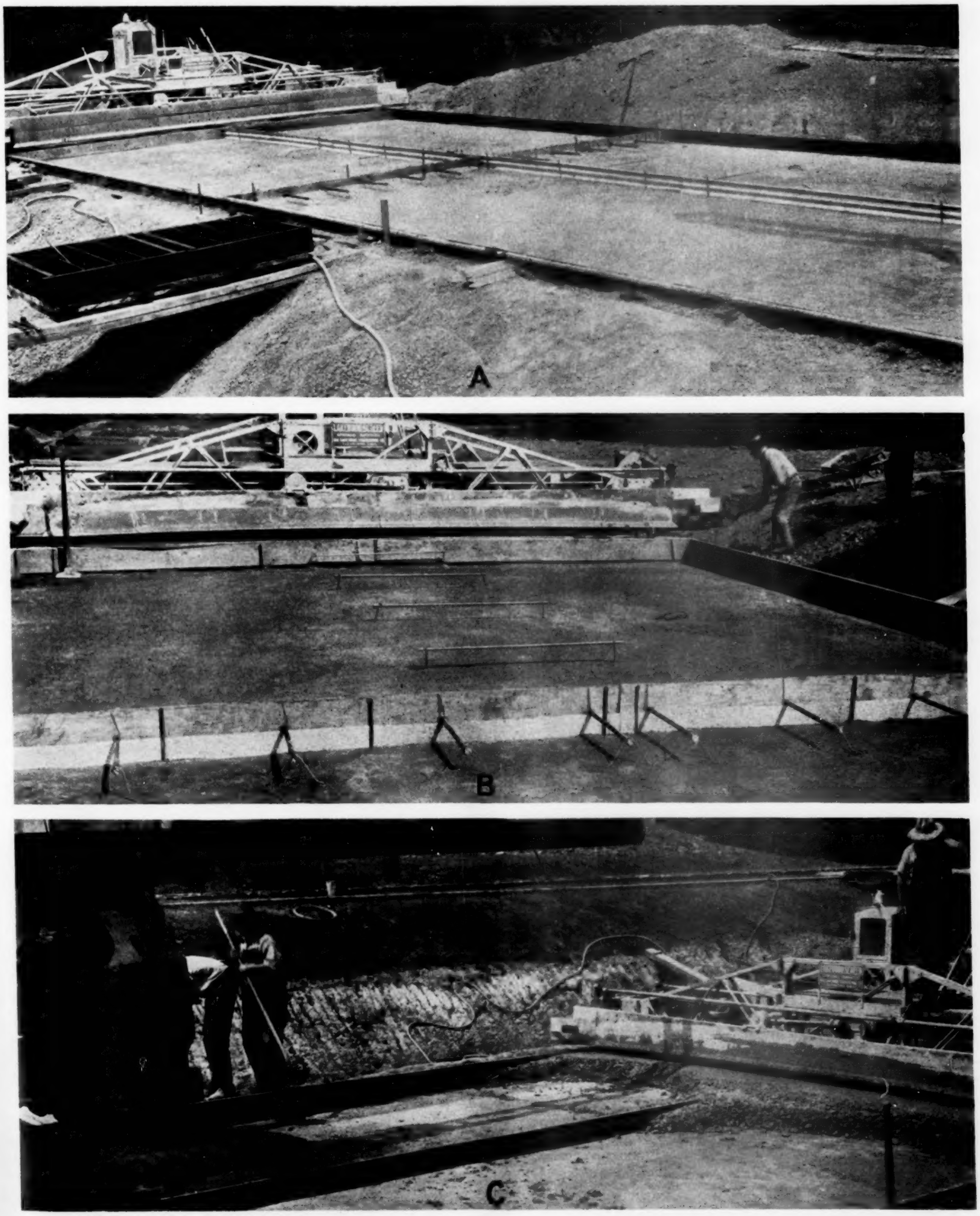
Details of the designs of the 10 longitudinal joints are shown in figure 7. With the exception of those found in sections 4 and 7, where no steel crosses the joint and free contraction is permitted, all of the designs are primarily warping joints.

By definition a dowel may or may not be bonded to the two pieces which it joins. In woodworking practice, dowels are more often bonded than not. In concrete pavement construction the dowels that cross the longitudinal joint are nearly always bonded to the concrete and are usually called tie-bars, although they are more exactly described as dowels in bond when they are called upon to withstand shearing forces. In this report the steel bars used for joining two abutting slab edges are generally referred to as dowels if the bond has been broken and dowels in bond if the bond still exists.

The longitudinal joint designs included in this investigation can also be grouped according to type.

The first group consists of four sections (sections 5, 3, 4, and 10) in which a tongue-and-groove construction was obtained by casting the concrete around a preformed, steel joint plate. The rectangular- and triangular-shaped tongue and groove and the sinusoidal tongue and groove (sections 3, 5, and 10, respectively) are held together with dowel bars in bond at 60-inch intervals.

In the second group are the longitudinal plane-of-weakness or dummy joints in which the surface of the slab was grooved to a depth of approximately one-third of the slab thickness at the time of construction, it being intended that an irregular fracture would subsequently develop extending from the bottom of the groove downward to the bottom of the slab. One of these sections (section 6) has dowel bars in bond placed



LONGITUDINAL AND TRANSVERSE JOINTS IN PLACE READY FOR THE CONCRETE TO BE CAST. A, CORRUGATED METAL PLATE USED TO FORM THE LONGITUDINAL JOINT IN SECTION 10. B, DOWELS IN PLACE FOR A DOWELED TRANSVERSE JOINT AND FOR A LONGITUDINAL WEAKENED-PLANE JOINT WITH DOWEL BARS IN BOND. THE WOODEN HEADER WAS LEFT IN PLACE UNTIL THE CONCRETE HAD HARDENED AND WAS THEN REMOVED. C, RECTANGULAR TONGUE-AND-GROOVE LONGITUDINAL JOINT.

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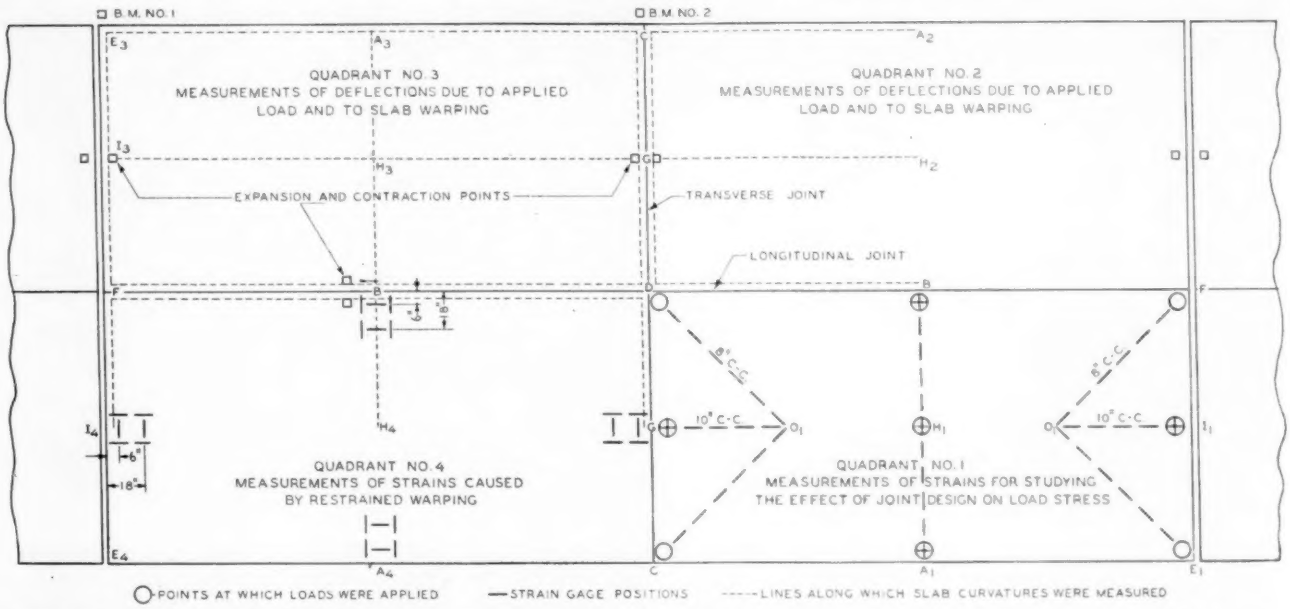

Figure 8.—Plan of a 20 by 40-Foot Test Section Showing the Positions of the Applied Loads and of the Measuring Instruments for the Study of Joint Behavior. Measurements of Deflections in Quadrants 2 and 3 Were Made With Loads Applied to Quadrant 3 at Load Positions Corresponding to Those Indicated by Circles in Quadrant 1.

across the joint at 60-inch intervals, while in the other (section 7) no dowels were used.

The third group embraces four sections (sections 1, 2, 8, and 9) in which the vertical faces of the abutting edges of the two 10-foot slabs were separated by a single thickness of tarred felt but held together by dowels in bond. These dowels were deformed bars of steel ¾ inch in diameter and 4 feet in length. They were spaced at intervals of 60, 48, 36, and 24 inches, respectively, in the four sections listed above.

As stated previously, in making the study of the structural behavior of these joint designs, tests were made on each to determine:

1. How freely expansion and contraction occurred.
2. To what degree unrestrained warping of the slab edges was permitted.
3. Their relative effectiveness in reducing the natural weakness of the joint edge by transferring load or by other means.

Measurements of expansion and contraction, of slab shape, and of slab deflection and strain under load were necessary to make these determinations. The location of the points at which the various measurements were made are shown on the plan of one of the test sections in figure 8. In this figure a letter is assigned to a definite point on a test slab while the subscript indicates the quadrant number. For example, the letter "E" is assigned to the free corner, and "H" the center point of the test panel and the subscripts 1, 2, 3, and 4 indicate the four quadrants of the test section. As in earlier descriptions, the various tests are described as having been made on the different quadrants of the test section for the sake of clarity in presentation. Actually certain tests were frequently made on more than one quadrant of a given test section.

SCHEDULE OF DEFLECTION AND STRAIN MEASUREMENTS OUTLINED

The points at which the expansion and contraction of the slab as a whole were measured are shown in the free ends and at the transverse joint in quadrants 2 and 3. The measurements were made with the specially constructed micrometer described in the first report of this series, the normal distance between the gage points being approximately 7 inches. The movements at the transverse test joint were compared with those at a joint which was known to be free to expand and contract and this comparison served as a basis for estimating the relative restraint offered by the various designs to longitudinal slab movements. These measurements covered both the daily and annual cycles of changes in slab length.

The study of the annual variations was made from measurements made twice daily over a period of about 2 years. The measurements were made at about 9 a. m. and about 3.30 p. m.

In the study of the daily variation in slab lengths, measurements were made on selected days at 2-hour intervals for a complete 24-hour period. The days were selected so as to give as wide a temperature range as possible for the particular season of the year.

From the data obtained it is possible to estimate very closely the extent of both of these cycles of change in slab length and joint width.

The relative restraint to free warping developed by the various joints was determined by comparing the magnitude of the deflection at the joint in question with that at a free edge under a given temperature condition and also by comparing the strains resulting from warping restraint at corresponding points at the free edge and at the joint under test. The shape of the deflected slab was determined with the clinometer and the movements of the extreme corners of the slabs were also measured with micrometer dials on fixed supports. The tests were usually started very early in the morning and readings were taken at 1-hour intervals until the maximum warping in each direction had occurred. In making the comparison for a transverse joint, deflections at the free corner (point E) were compared with those at the transverse joint corner (point C). For a longitudinal joint the deflections at the free corner

(point E) were compared with those at a longitudinal joint corner (point F).

The lines along which the clinometer points were installed for the warping studies are shown in the third quadrant of figure 8. The restraint to warping offered by a transverse joint was indicated by a comparison of the curvature along the line E_3—A_3 with that along the line C_3—A_3. For the same study of the longitudinal joint action the curvature along the line E_3—I_3 was compared with that along the line F_3—I_3.

In measuring the curvature with the clinometer two sets of readings were made for each comparison, the first in the early morning at a time when the upper surface of the pavement was at a lower temperature than the lower surface and a second set in the early afternoon when the temperature of the upper surface of the slab was above that of the lower surface.

Since any tendency of the joint to restrain the slab edge from warping freely would be reflected by an increase in the magnitude of the warping stresses, a comparison was made in each case between the warping stresses at a free edge and those at the joint in question. The method of arriving at the values of the warping stresses from measured strains was described in part 2 of this series of reports. The location of the strain gages for these comparisons is shown in the fourth quadrant of the test section in figure 8. In the study of the transverse joint the stresses indicated by the group of gages at I_4 were compared with those measured by corresponding gages at G_4. Similarly, for the study of the longitudinal joint the stresses indicated by the group of gages at A_4 were compared with those found at the corresponding positions at point B_4.

As remarked before, every joint is potentially a structural weak spot and some means for strengthening this part of the slab is usually a part of the joint design. The common method is by transferring part of the load to the adjoining slab through the shear resistance of the joint. In this investigation the relative effectiveness of the various joints from the standpoint of their ability to strengthen the slab edge was determined by comparing the critical strains and deflections caused by a load acting near a joint edge with those produced by the same load at other points on the slab.

Loads were applied at the four corner points C, D, E, and F (fig. 8) to determine how effectively the joint functioned in reducing the critical deformations caused by a load acting at the corner of the slab. Similarly, the effectiveness of the design under the action of loads applied at the joint edge (but away from the slab corner) was determined from data obtained with loads applied at points A, B, G, I, and H. The lines along which the curvature of the slab was measured under the action of the applied loads are shown in the second, third, and fourth quadrants, while the strain-gage locations that were used in this part of the study are shown in the first quadrant of this figure. The detailed schedule of the deflection measurements follows:

Load applied at point:	Deflection measured along line:
CORNER TEST	
E_3.	E_3—A_3 and E_3—F_3.
C_3.	A_3—A_2 and C_3—D_3.
F_3.	E_3—I_4.
D_3.	D_3—B_3.
EDGE TEST	
A_3.	A_3—B_3.
B_3.	A_3—H_4.
I_3.	I_3—H_3.
G_3.	H_3—H_2.

For each test the shape of the unloaded slab was determined, the load was applied and the shape of the loaded slab measured, the change in shape being taken as the deflection caused by loading.

CRITERION OF JOINT EFFICIENCY ADOPTED

A somewhat similar schedule was followed in making the strain measurements. For the load applied at the slab corners the strains were measured in the upper surface of the slab along the bisector of the corner angle. For example, with a load applied at point E_1 the strains were measured along the line E_1—O_1 and similarly for the other corners of the slab.

For loads acting at the edge points the strains were measured both parallel and perpendicular to the slab edge at the point of load application and for a sufficient distance along a line perpendicular to the edge to insure the finding of the critical tensile stress in the upper surface of the slab. For example, with a load acting at point A_1 the strains were measured in both directions directly under the load and along the line A_1—H_1.

Loads were applied at point H solely for the purpose of obtaining a comparison of the critical stresses caused by a load at this point with those caused by the same load applied at certain other points. Since the critical stresses occur directly under the load in the case of a load acting in the interior of the slab, only the strains developed in the upper surface directly under the load were measured. These strains at point H were measured in both the longitudinal and transverse directions.

Before making a comparison of the relative effectiveness of various joints for accomplishing any certain purpose, it is necessary to establish some rational basis of comparison. If it is desired to compare joints on the basis of their ability to reduce the deflection of the slab edge at some particular point, then deflection measurements at that point may be used as a means for estimating the effectiveness of the joint. However, if the purpose of the joint design is to reduce the stresses from applied loads so as to, in effect, increase the load-carrying capacity of the edge of the slab, then it is necessary to arrive at the basis of comparison through the measurements of strains and not deflections, for it has been clearly demonstrated in these tests that the precision of the deflection data is not sufficient to warrant any conclusions relative to attendant stress conditions. The question then arises as to how stresses determined from strain measurements may best be used as a basis for judging the relative structural effectiveness of various joint designs.

It has been established that if a given load is applied at various points on the surface of a concrete pavement slab of uniform thickness the critical stress will be a minimum when the load is applied at an interior point and that the critical stress will reach its maximum value when the load is applied at the free edge.

If a joint operated with a maximum amount of structural efficiency, it would reduce the critical load stress at the joint edge to a value equal to that found in the interior of the slab. If, on the other hand, it was completely ineffective the critical stress for a load at the joint would equal the critical stress for the same load acting at a free edge.

These two values therefore, delimit the range of joint efficiency so far as the ability to reduce load stresses is concerned and suggest a stress formula which will furnish a rational measure of joint efficiency.

If, for a given applied load on a slab of uniform thickness,

σ_j is the critical stress for the load applied at the joint edge,
σ_f is the critical stress for the load applied at the free edge,
and σ_i is the critical stress for the load applied at an interior point.

Then the joint efficiency, E, may be expressed as follows:

$$E=\frac{\sigma_f-\sigma_j}{\sigma_f-\sigma_i}$$

In other words, the reduction in edge stress accomplished by the joint under consideration is compared to that accomplished by the complete continuity of the interior condition, as a measure of efficiency.

In making the stress determinations upon which the joint efficiency values were based, it was not considered desirable to depend entirely upon stress values obtained at a single point no matter how well established the value might be.

For the determination at each load point, therefore, eight tests were made, each at a somewhat different location. For example, to establish a stress value for the free edge (point A) eight tests were made altogether, and in no two was the bearing p ate in the same spot on the same quadrant of the test section, although in all cases it was at the free edge and close to the midpoint of the slab length.

Tests were made also at all of the longitudinal joint corners on the four constant-thickness slabs but were not made at these corners on the thickened-edge slabs because there would be no basis for comparing strains measured at corners of different thicknesses.

Deflection and strain data that indicate the strengthening effect of thickened edges at slab corners appear later in this report.

DATA ON VARIATIONS IN JOINT WIDTH PRESENTED

The annual variation in the widths of the various transverse joint openings is indicated by the data shown in figure 9 in which the ordinates are the variations in the measured joint width when compared to a set of initial measurements made in November 1930, shortly after the pavement was constructed. The morning measurements were made between 9 and 9:30 and the afternoon measurements between 3:30 and 4 o'clock.

The joint designations used in this figure are as follows: The transverse joint in the center of the test section is given the same number as the test section in which it is located. For example, joint 3–3 refers to the transverse joint across the center of test section 3. The open joint between two adjoining test sections is given the numbers of the two sections between which it is found, as for example, joint 2–3 is the open joint between test sections 2 and 3. It will be recalled that these open joints between the test sections were all 2 inches wide and were kept open, preventing any connection between the slab ends.

It was mentioned earlier that the expansion and contraction measurements were of necessity made at the level of the upper surface of the pavement. The observed horizontal movements were therefore the results of the direct expansion and contraction of the slab combined algebraically with the horizontal component (at the plane of the measurements) of any tilting of the slab ends caused by warping. To determine the changes in joint width caused solely by expansion or contraction of the slab, it is necessary to correct the observed changes for the effect of the warping present at the time of observation.

Figure 9 shows the seasonal variations in observed joint width. The correction for warping mentioned in the preceding paragraph is considerably more important in a consideration of the daily variations than it is when seasonal changes are involved for the following reasons:

The afternoon observations were made at the time of maximum downward warping of the slab edges. During the summer when the pavement was expanded to its greatest length the slab edges were warped upward by moisture conditions within the slab to the maximum degree. Data were presented in the second report of this series to show the relative degrees of the daily and seasonal warping found in the test sections at different seasons of the year. The data presented in that report on the effect of seasonal moisture changes indicated that the moisture condition was rather unstable during the summer months. This resulted from abnormal weather conditions during the summer of the year in which the observations were made. Moisture warping observations that have been made since that report was written and the data showing the change in length of the pavement caused by moisture changes both indicated that, under normal summer weather conditions, the moisture state of the pavement is quite stable during the summer months.

It is indicated by the data just mentioned that the downward warping of the slab edges on critical days during the summer at the time of the afternoon observations is approximately equal to the upward warping caused by the seasonal change in its moisture condition. This conclusion is based upon the assumption that all of the seasonal moisture warping is upward; in other words, that there is at no time more moisture in the pores of the concrete in the upper part of the pavement slab than is present in those near the subgrade. This assumption seems to be supported by all of the data available from these tests.

It appears therefore that, at the time the measurements indicating the maximum closing of the joints were made, the slabs were probably in nearly a flat condition. For the morning observations in winter all evidence indicates little warping from either moisture or temperature. If these assumptions are correct, data such as are shown in figure 9 indicate the seasonal variations in the widths of the joints with sufficient accuracy without corrections for the effects of warping. The indicated daily variations in joint width are not true measures of expansion and contraction and should not be used without correction. The method used for determining the magnitude of the warping correction is described a little farther on in this report in connection with the discussion of daily variations in joint width.

SLABS DID NOT EXPAND AND CONTRACT SYMMETRICALLY

If a comparison is made between the annual variations in width of the several joints, it will be observed that the movements at the joints formed by the free ends of the slabs are, in general, greater than those at the regular transverse joints. The observed difference between the opening of the transverse test joints and that of the open joints between the test sections indicates that the resistance of the former to expansion and contraction movements causes either a deformation that alters the length of the slab or a shifting of the center of the slab panel longitudinally over the subgrade.

That there is no appreciable stress deformation that changes the slab length is shown by the numerous

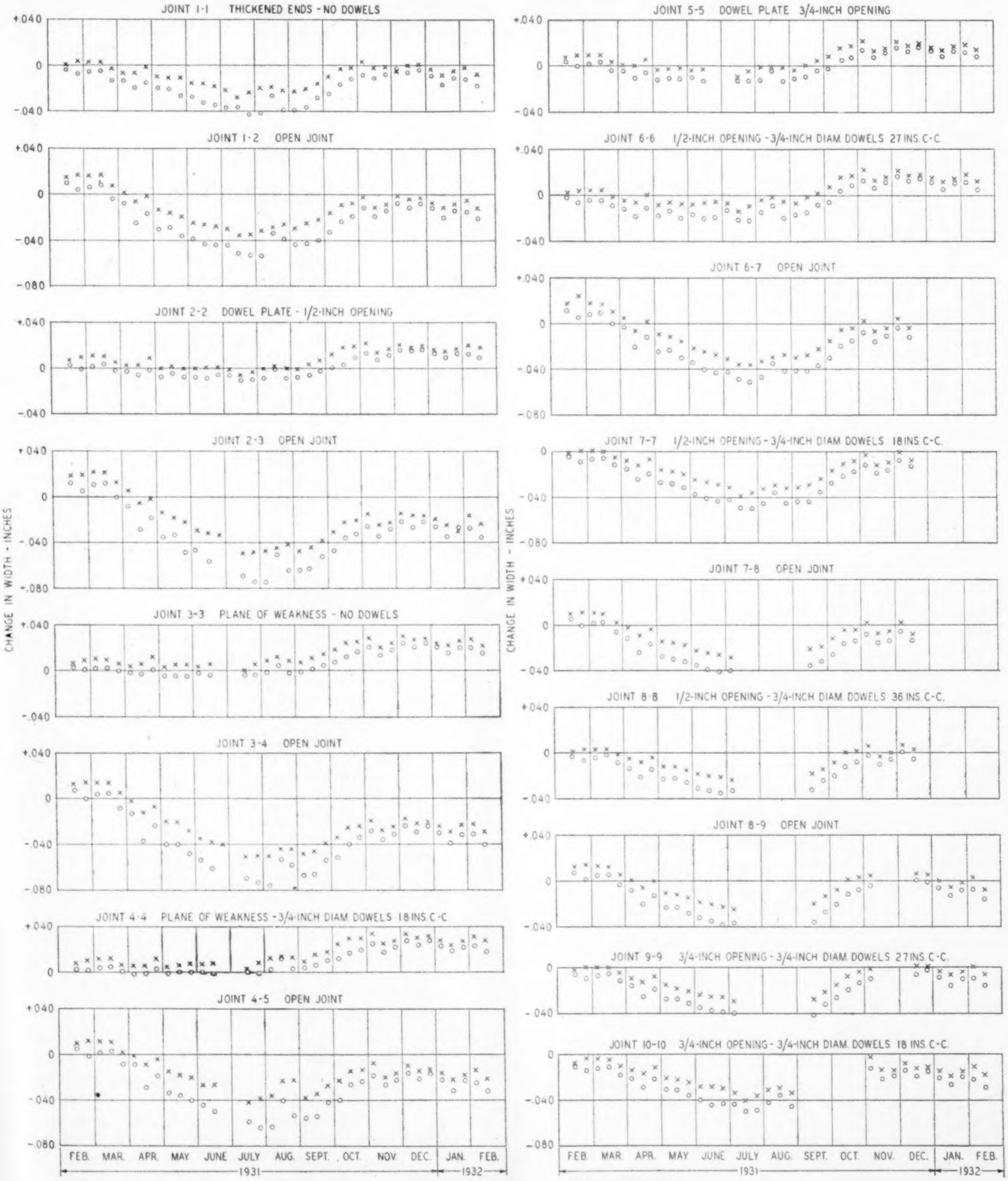

FIGURE 9.—SEASONAL VARIATIONS IN WIDTH OF EACH OF THE TRANSVERSE JOINTS OVER A TYPICAL 1-YEAR PERIOD. JOINT OPENING SHOWN AS POSITIVE, AND JOINT CLOSING SHOWN AS NEGATIVE. EACH VALUE IS A 10-DAY AVERAGE.

measurements of the variation in slab length with temperature changes that were presented in the second paper of this series. It will be remembered that these indicated that the deformation or change in slab length caused by the subgrade resistance during expansion or contraction is negligible in slabs of this length. It must be concluded, therefore, that the 10- by 20-foot slabs do not expand and contract symmetrically with respect to the subgrade at their midpoints. This eccentricity of movement is evident in all of the sections, although it is more noticeable in some than in others. It will be discussed in more detail a little farther on in this report.

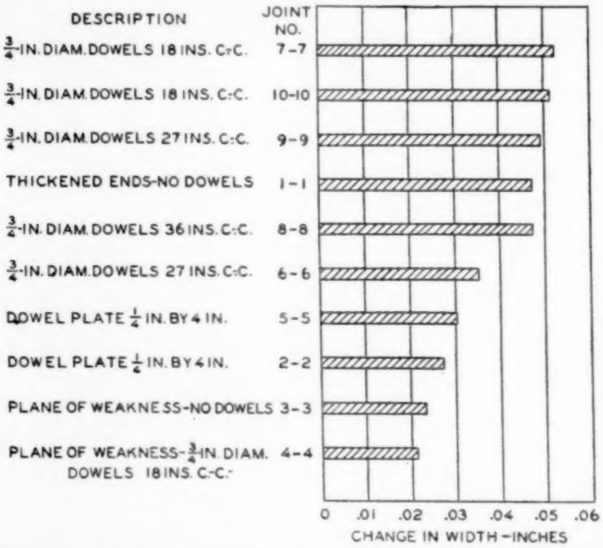

FIGURE 10.—MAXIMUM SEASONAL CHANGES IN WIDTH OF THE TRANSVERSE JOINTS. MAXIMUM RANGE FEBRUARY 16 TO JUNE 22, 1931. THE RANGES SHOWN ARE FROM THE AVERAGE MINIMUM AS SHOWN BY THE MORNING MEASUREMENTS ON COLD DAYS IN FEBRUARY TO THE AVERAGE MAXIMUM AS SHOWN BY THE AFTERNOON MEASUREMENTS ON HOT DAYS IN JUNE.

It will be observed further that several of the transverse joints opened more during the winter of 1932 than during the winter of 1931 and that consistently the adjacent open joints opened less during this same period and each by approximately the same amount. This condition is most noticeable in the two transverse planes of weakness (joint 3–3 and joint 4–4) and for the two joints containing the dowel plates (joints 2–2 and 5–5), and also, for some reason that is not apparent, in the dowelled joint 6–6. During 1931 these joints closed very little if any after March.

Figure 10 was constructed, from the same basic data that were presented in the preceding figure, for the purpose of showing the relative freedom of the different transverse joints to expand and contract. Selecting arbitrarily the period between February 16 and June 22 as giving a wide range in temperature, the average maximum range of movement for each of the transverse joints during this period was determined. In the preparation of figure 10, the daily values rather than 10-day averages such as are shown in figure 9 were used to obtain the average maximum and average minimum values. The indicated seasonal movements are therefore greater in figure 10 than in figure 9. The data for the joints are arranged in this figure in the order of descending values of the observed maximum seasonal movement. Since the sections are all of the same length and each is completely separated from its neighbors, the amount of movement which occurs at each test joint during a given period of time may be assumed to be a measure of the relative freedom of action so far as expansion and contraction are concerned.

Joint 1–1 was constructed as a clear opening one-half inch wide between two thickened-end slabs. It was filled with a bituminous joint filler shortly after construction. So far as the joint filler is concerned, it should offer relatively little resistance to expansion and contraction movements. The opinion has been expressed that this type of slab end exercises a restraining action that prevents the slab from contracting freely and causes a corresponding stress in the concrete, due to the inclined surface of the subgrade over which it presumably has to move.

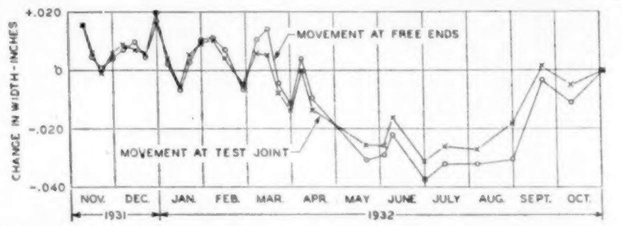

FIGURE 11.—COMPARATIVE MOVEMENTS AT THE TRANSVERSE JOINT AND AT THE FREE ENDS OF ONE OF THE TEST SECTIONS. POSITIVE VALUE INDICATES OPENING OF THE JOINT AND NEGATIVE VALUE INDICATES CLOSING OF THE JOINT.

In the second report of this series it was shown that, for the subgrade material and slab lengths concerned in these tests, the earth of the subgrade adhered to the bottom of the slab, "bending" or moving forward with the slab. Under such conditions the incline of the lower surface of the slab would not increase the resistance over that of a flat slab. It is indicated by the data in figure 10 that the thickened-end slab joint 1–1 permits the slabs to expand and contract as freely as any of the other transverse joints tested in this investigation.

Joints 7–7, 10–10, 9–9, 8–8, and 6–6 contain the unbonded, round, steel dowel bars. The movements at all of these are of approximately the same magnitude and about the same as that at joint 1–1, with the one exception of the seasonal movement of joint 6–6. There is no apparent reason why the seasonal movement of this one joint should be appreciably different from those of the other joints of the same type. The data indicate a high degree of relative freedom for the dowel joints with little or no variation in the restraint with the number of dowels per joint.

Other data, which supplement those shown in figure 9, were obtained in the measurements on section 10. These data are given in figure 11 in which the amount of movement at the test joint 10–10 and at the free ends of the section are shown at frequent intervals over a period of about 1 year. This section was separated from the one adjoining by an open space of considerable width. The expansion and contraction measurements were therefore made to fixed reference points at each end of the section. Since both ends were completely free the difference between the movements at the free ends when compared with the corresponding movements at the transverse test joint, as shown in figure 11, give a good idea of the degree of restraint developed in joint 10–10. In figure 10 this joint is among the group compared, hence a basis is furnished for estimating the order of restraint to expansion and contraction offered by each of the joints.

DOWEL-PLATE JOINTS OFFERED MORE RESISTANCE TO SLAB MOVEMENT THAN DID UNBONDED DOWEL BARS

For example, if the maximum movement found during the year shown in figure 11, i. e., November 1931 to November 1932, for joint 10–10 is expressed as a percentage of the movement measured at the completely free slab ends, it will be found that the movement at the joint was, in round numbers, 80 percent of that at the free ends of the test section. If this percentage is applied to the movement shown in figure 10 for joint 10–10, the analysis indicates that a movement for a completely unrestrained joint would be of the

order of 0.065 inch. With this value as a basis, the restraining action of each of the 10 joint designs can be estimated.

It is indicated that joint 1–1, constructed as an open joint and filled with a poured joint filler, is not completely free and it seems likely that during the expansion of the concrete the compression of the filler material in the joint required an amount of force approximating that required to overcome both the resistance of the filler and the resistance of the dowels in each of joints 7–7, 10–10, 9–9, and 8–8. If this is so, then the force required to cause the dowels to slide in these joints must be very small, because the resistance of the joint filler to compression must be about the same in each of the joints in this group.

Joints 5–5 and 2–2 contained the one-fourth by 4-inch steel dowel plates. The measurements show that the seasonal movement of these two joints was approximately three-fifths of that of the dowelled-joint group (7–7, 10–10, 9–9, 8–8, and 6–6). It is indicated, therefore, that the plate-dowel joints offer a greater resistance to expansion and contraction than do joints containing properly installed round steel dowel bars which are not in bond with the concrete.

Joints 3–3 and 4–4 are the two transverse plane-of-weakness or dummy-type joints; the latter contain three-fourths-inch diameter dowel bars spaced 18 inches between centers. The seasonal movements of these joints are the smallest for any of the transverse joints. That this is caused by the complete closing of these joints in the early summer is clearly shown by the data already presented in figure 9. In examining this figure, attention is called particularly to the large movements of joint 3–4 lying between the two dummy joints. The closing of the dummy joints throws any subsequent expansion into slab displacement or slab deformation under stress. In such slabs the short slab length and the adjacent open joints provide the necessary relief from compression. The plane of weakness joints contract freely and relieve tension. Also, joints of this type undoubtedly reduce greatly the warping stresses. They do not appear to relieve slab expansion to any great extent, however.

Figure 12 shows data obtained during a cycle of measurements of width made at each of the transverse joints at approximately 2-hour intervals over a period of 24 hours. As noted in the figure, these particular observations were made during the latter part of June 1931, the time of year when large temperature differentials are developed in the test slabs.

On the same day that the daily variations in transverse joint width shown in figure 12 were obtained, the warped shapes of the slabs were measured at intervals of approximately 2 hours with the clinometer. From these clinometer data, curves similar in character to those shown in figures 25–28 and 31 of the second paper of this series were obtained. These curves were plotted to suitable scales and the slope of the upper surface at the extreme end of the pavement slab was estimated graphically. From this the change in slope of the vertical end face of the slab was determined for the rather extreme conditions of temperature warping which obtained on the particular day that the measurements were made. From this change in slope the effect upon the measurement of joint width was calculated and applied as a correction to the expansion

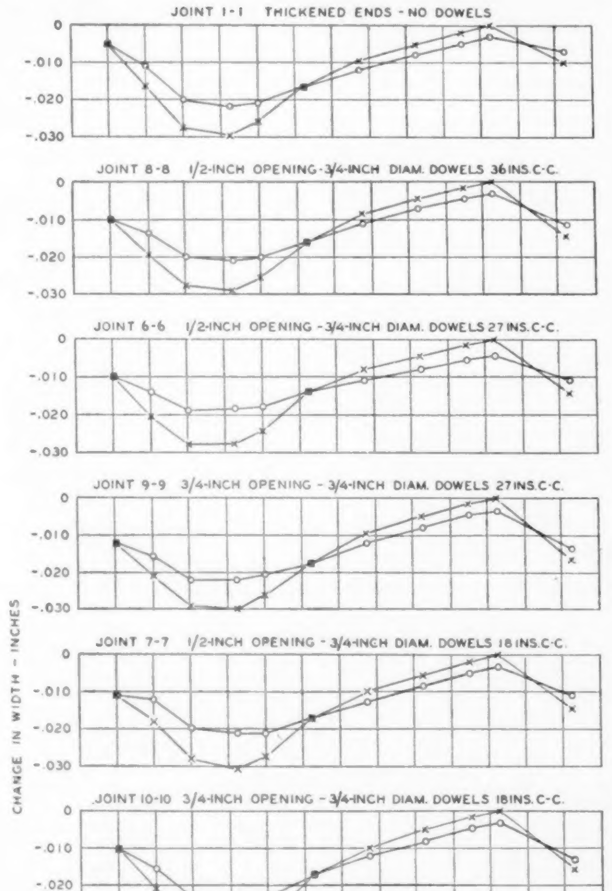

Figure 12.—Measured Changes in Width of the Various Transverse Joints Caused by Daily Changes in Temperature of the Pavement. Negative Values Indicate Closing of the Joint.

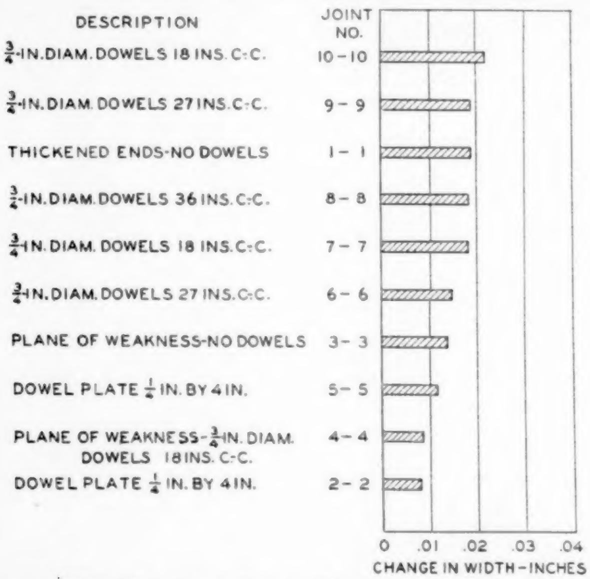

FIGURE 13.—MAXIMUM DAILY CHANGES IN WIDTH OF THE TRANSVERSE JOINTS.

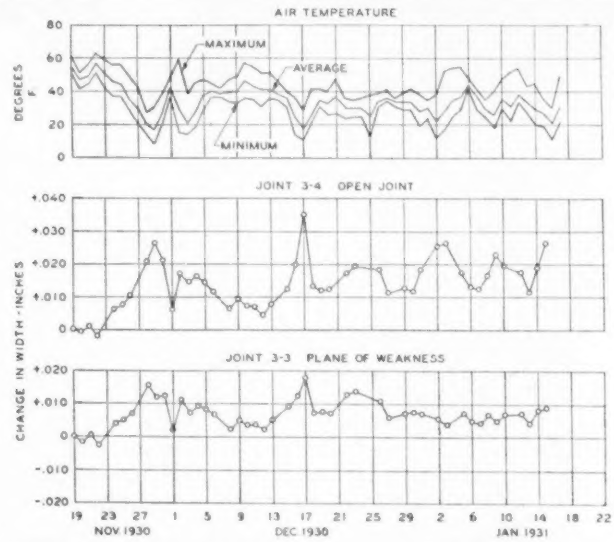

FIGURE 14.—CHANGES IN THE WIDTH OF ADJACENT TRANSVERSE AND OPEN JOINTS DURING A PERIOD WHEN ECCENTRIC MOVEMENTS WERE NOTED. OBSERVATIONS WERE MADE AT 2 P. M. POSITIVE VALUE INDICATES OPENING OF THE JOINT AND NEGATIVE VALUE INDICATES CLOSING OF THE JOINT.

and contraction measurements obtained at the transverse joints on the same day. Both the uncorrected and the corrected values are shown in figure 12.

It will be noted that the apparent change in joint width caused by the rather extreme temperature warping amounts to a closing of 0.007 to 0.009 inch during the day and an opening of 0.003 to 0.004 inch during the night. From the corrected curves of daily variations in joint width, figure 13 was constructed to show the relative extent of the true expansion and contraction that occurred at each transverse test joint for this particular June day.

In general the indications of figure 13 as to relative joint freedom are the same as those shown in figure 10 for the slower seasonal movements. The restraint to expansion and contraction offered by the dowel-plate joints is apparent in figure 13 and it appears that this restraint is greatest at the time when the slab edges are warped downward to the greatest degree. It is possible that slab warping causes an increase in the friction between the plate and its sockets and that the irregularity in these curves is caused by the variation in the frictional resistance as the extent of the slab warping varies.

ECCENTRICITY OF SLAB MOVEMENT DURING EXPANSION AND CONTRACTION STUDIED

In connection with the discussion of figure 9, mention was made of a tendency for the measured movement at the transverse test joints to differ in magnitude from that observed at the open joints at certain times and under certain conditions of temperature. A study of this relation has been made throughout the period of the investigation and much has been learned of its nature although the causes for the observed behavior are not entirely clear.

Figure 14 shows the movements at joint 3–3, a transverse plane of weakness without dowels. Figure 14 also shows the corresponding movements at the open joint 3–4 and maximum, minimum, and average air temperatures for a period of about two months during the winter of 1930–31.

It will be noted in this figure that frequently the movement at the test joint is of about the same magnitude as that at the free end, and yet whenever a marked temperature change occurs there is a tendency for the movement at the free end greatly to exceed that at the test joint. This effect is particularly noticeable during a sudden drop in temperature such as that which occurred between December 15 and 17, 1930. Whenever the observed movement at the free end exceeds that at the transverse joint, the slab is not expanding or contracting symmetrically with respect to the midpoint between the two, the slab panel being shifted as a whole toward the end at which the smallest change of position was observed.

This phenomenon was noticed on all of the test sections, being greater on some than on others, and being noticeably greater during the winter than in the summer. Also, it was greater during the first winter following the laying of the pavement than during the second winter, as can be seen by an examination of the data in figure 9.

To bring out the variation in the degree of eccentricity during the year, figure 15 was prepared from observations on section 7. The transverse joint in this section contained round dowels at 18-inch intervals. The movements of the transverse joint, expressed as percentages of that at the free end of the slab are shown as ordinates to the curve. It is interesting to note how this variation follows in a general way the annual variation in average daily air temperature, definitely indicating that the phenomenon is caused primarily by temperature. During the summer months the movements at the two points of measurement are nearly the same, but during cold weather the movement at the transverse joint is from 5 to 15 percent less than that at the open joint at the free end of the slab. As already stated, the difference was found to be greatest, for any particular season, at times of sudden change in temperature.

The observations were continued over a period of nearly 5 years in order to determine whether the movements tended either to open or to close the transverse joints or whether they were compensating in their effects.

The net change in width of each of the transverse joints and of each of the open joints on which measurements were made is shown, for the period between November 1930 and August 1935, in table 2.

TABLE 2.—*Changes in transverse joint width* [1] (*November 1930 to August 1935*)

Test joints		Open joints	
Joint no.	Net change in width	Joint no.	Net change in width
	Inches		*Inches*
1-1	+0.010	1-2	−0.074
2-2	+.096	2-3	−.089
3-3	+.093	3-4	−.108
4-4	+.110	4-5	−.090
5-5	+.072	5-6	
6-6	+.039	6-7	−.026
7-7	+.003	7-8	−.034
8-8	+.018	8-9	−.023
9-9	+.005	9-10	
10-10	−.024		

[1] The values shown are the net changes in joint width after the observed widths had been corrected for the estimated effects of pavement temperature difference and moisture difference, averaged for the two halves of the pavement (on either side of the longitudinal center joint) and expressed in inches per 20 feet of slab length. A positive sign indicates an opening and a negative sign a closing of the joint.

The data indicate that while all of the transverse joints, except that in section 10, opened to some degree, the two weakened-plane joints (3–3 and 4–4) and the two containing the dowel plates (2–2 and 5–5) apparently opened by about one-tenth inch during the period covered by the observations. In all of the other sections the opening has been very much less and in section 10 a slight closing has apparently occurred.

A comparison of the movement at the transverse joints with that at the open joints will show that the change in width has been accompanied by a corresponding change in the open joints. This indicates that some resistance to expansion and contraction existing at the transverse joint caused a permanent displacement of the pavement slab away from the transverse joint and toward the open joint. The magnitude of the displacement is probably a rough measure of the relative freedom of the various transverse joints to expand and contract.

In figure 20 of the second paper of this series data were presented to show the approximate magnitude of the force required to move a slab a given distance on this particular subgrade. An estimate of the restraining force developed by the different transverse joints may be made by finding the distance through which any particular joint causes the abutting slab to be displaced over the subgrade and then calculating the force required to cause this movement.

LONGITUDINAL JOINTS TENDED TO OPEN IN WARM WEATHER

The restraining forces were calculated in this manner for the four sections in figure 10 that show the least seasonal movement at the transverse joint (secs. 2, 5, 3, and 4), and the estimated unit stress developed by joint restraint in each is shown in table 3 with the data upon which the calculations were based. It is indicated that the dowel-plate joints caused restraint which may develop either tensile or compressive stresses of approximately 30 pounds per square inch, and the plane-of-weakness or dummy joint may cause compressive stresses of approximately the same magnitude. It is probable that if the slabs had been forced to expand and contract against the full resistance of their joints, instead of being comparatively free to shift their position as they were in these tests, a much more serious stress would have resulted.

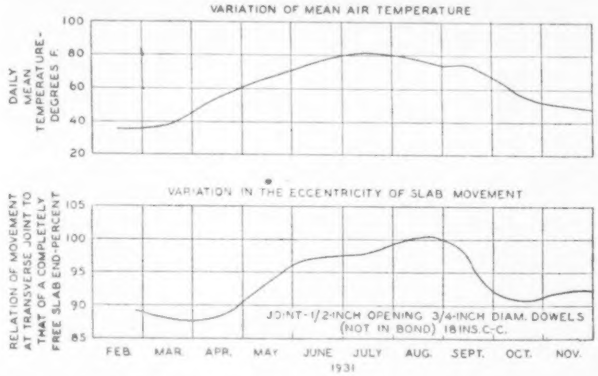

FIGURE 15.—VARIATIONS IN THE ECCENTRICITY OF SLAB MOVEMENT COMPARED WITH THE VARIATIONS OF MEAN AIR TEMPERATURE. THE CURVES SHOW THE GENERAL TREND OF THE TEMPERATURE AND OF THE ECCENTRIC MOVEMENTS OF THE JOINTS BUT DO NOT INDICATE CHANGES THAT TAKE PLACE OVER SHORT PERIODS OF TIME.

The tendency for the joints to increase in width with time is of particular interest in connection with the weakened-plane joints when no dowels or other positive means for load transfer has been provided. The increase, though small, is a large percentage of the width of the original crack and it makes the load-transfer action of such joints problematical. It also creates a joint opening that is difficult to seal against moisture and solid matter.

TABLE 3.—*Estimated stresses resulting from joint restraint during expansion and contraction* [1]

Test section no.	Type of slab cross section	Type of transverse joint	Weight of slab, 10-foot width	Displacement of slab	Total computed thrusting force	Area of cross section	Estimated unit stress [2]
			Pounds	*Inches*	*Pounds*	*Sq. in.*	*Lbs. per sq. in.*
2	9-7-9	Dowel plate	18,250	0.025	25,550	875	±29
5	9-6-9	do	16,125	.022	21,350	774	±28
3	9-6.3-9 (A. A. S. H. O.).	Plane of weakness (no dowels).	17,775	.029	27,100	853	−32
4	9-6.3-9 (parabolic).	Plane of weakness (doweled).	18,000	.031	28,800	864	−33

[1] The period covered is the same as that shown in fig. 10.
[2] A positive sign indicates tensile stress and a negative sign indicates compressive stress.

The daily measurements of width of the longitudinal center joints of 6 of the 10 test sections over a period of about 1 year are shown in figure 16. Joints C–6 and C–7 are not shown, as they were constructed as planes of weakness and had not cracked through at the time of the measurements. Joints C–3 and C–10 had not been equipped with measuring points at the time of the measurements.

The measurements were made at the same time of day as those at the transverse joints, so that this graph shows the same relation as was brought out in figure 9, i. e., the maximum seasonal movements. In studying this graph it is well to bear in mind that the effective slab length in this direction is 10 feet and that all joints except C–4 and C–7 were crossed by bonded steel bars 4 feet in length. As noted above, joint C–7 had not cracked through at the time of the measurements but joint C–4 affords a good example of a free joint for purposes of comparison. It will be observed that the movement of this joint is approximately twice as great as that measured at the bonded joints.

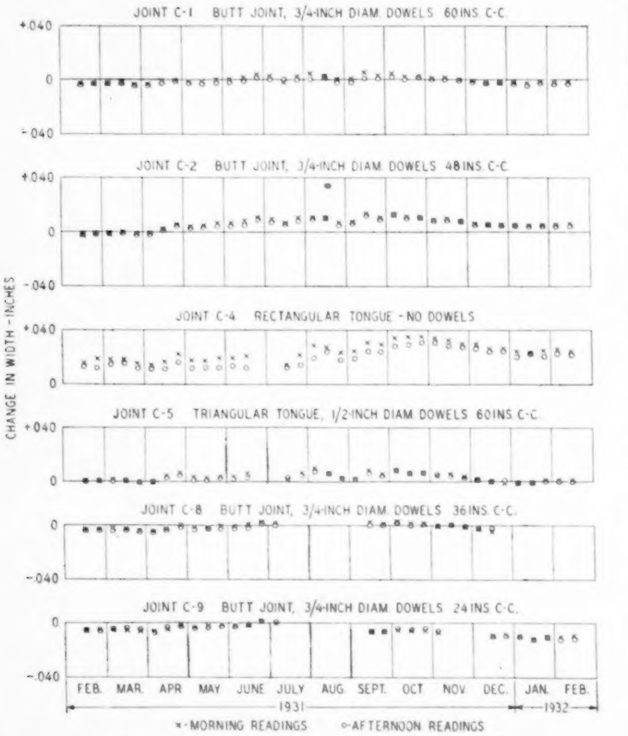

FIGURE 16.—SEASONAL VARIATIONS IN WIDTH OF EACH OF THE LONGITUDINAL JOINTS OVER A TYPICAL 1-YEAR PERIOD. JOINT OPENING SHOWN AS POSITIVE AND JOINT CLOSING SHOWN AS NEGATIVE. EACH VALUE IS A 10-DAY AVERAGE.

These data indicated that in all cases the longitudinal joints tend to open as warm weather comes on, although a slight closing occurs each day as the slab expands with temperature. The possibility that this rather curious behavior was the result of temperature and moisture warping was investigated in the same manner as that described in the discussion of transverse joint movements. It was found that the change in slope of the slab edge at the longitudinal joint under extreme conditions of daily temperature warping might be as great as 10/10,000 for maximum downward warping (afternoon condition) and 2/10,000 for maximum upward warping (night condition).

The effect of moisture change was also investigated and it was found that from extreme upward warping from this cause (summer condition) to extreme downward warping (winter condition) a change in slope of the slab edge of about 12/10,000 occurred. The normal or unwarped condition of the slab, representing the condition where no moisture gradient was present, was not determined so that it is not possible to break down this total change in slope into its two components of upward and downward warping. The evidence indicates, however, that the slab is practically unwarped from this cause during the winter.

CERTAIN TYPES OF TRANSVERSE JOINT OFFERED LITTLE OR NO RESTRAINT TO SLAB WARPING

Assuming that as warping occurs, joints containing dowel bars in bond either widen or close at the top of the slab and that at the plane of the bars the joint width remains constant, it is found that the total temperature warping would cause an apparent daily change in width of the longitudinal joint of a 7-inch slab (with the bonded dowel bars at mid-depth) of 0.008 inch. Since

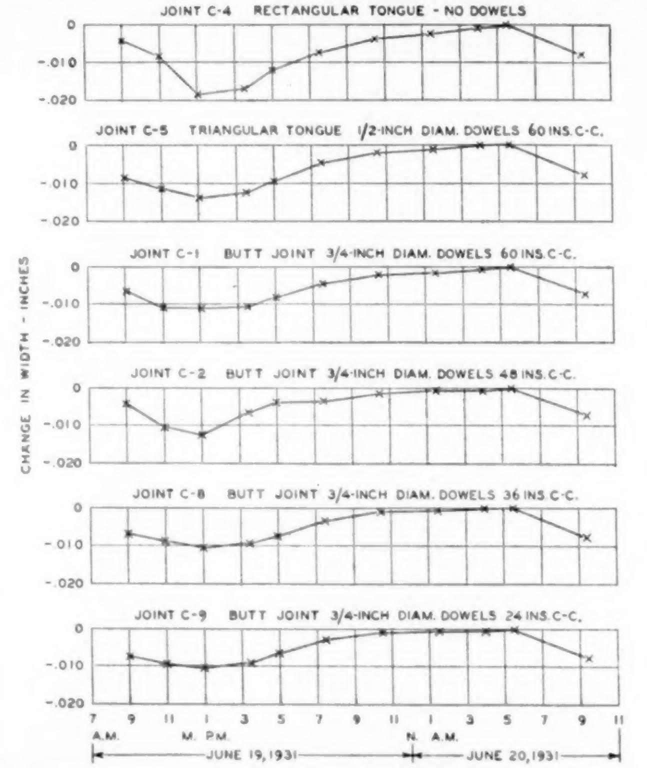

FIGURE 17.—APPARENT CHANGES IN WIDTH OF THE VARIOUS LONGITUDICAL JOINTS CAUSED BY DAILY CHANGES IN TEMPERATURE OF THE PAVEMENT. A NEGATIVE VALUE INDICATES CLOSING OF THE JOINTS.

the total change in slope for extreme daily temperature warping equals that for total warping caused by annual moisture changes, it is probable that moisture change will produce an annual change in width of approximately the same magnitude.

The effect of the daily cycle of temperature changes is an apparent closing of the joint during the day and a corresponding opening during the night. The seasonal cycle of moisture variations should cause an apparent opening of the joints during the summer months and a closing during the winter.

This is in agreement with the observed behavior, as will be seen by referring to figures 16 and 17, which show the uncorrected measurements of longitudinal joint width during a 24-hour cycle of temperature changes. It is found that the magnitude of the observed variations agrees closely with what might be expected from the warping that was known to occur.

While the method of computing the effect of warping on joint width is necessarily an approximate one, it is believed that these computations show quite definitely that a large part of the apparent variation of longitudinal joint width shown in figures 16 and 17 is caused by slab warping and that, in the case of the joints containing bonded steel, there was in reality little or no opening and closing caused by expansion and contraction.

Joint C–4 is a rectangular tongue-and-groove joint without bonded steel. There is a tendency for this joint to increase in width with time. It is not known how long this progressive opening will continue but the fact that it has opened is of particular interest as indicating the probable movement of the longitudinal

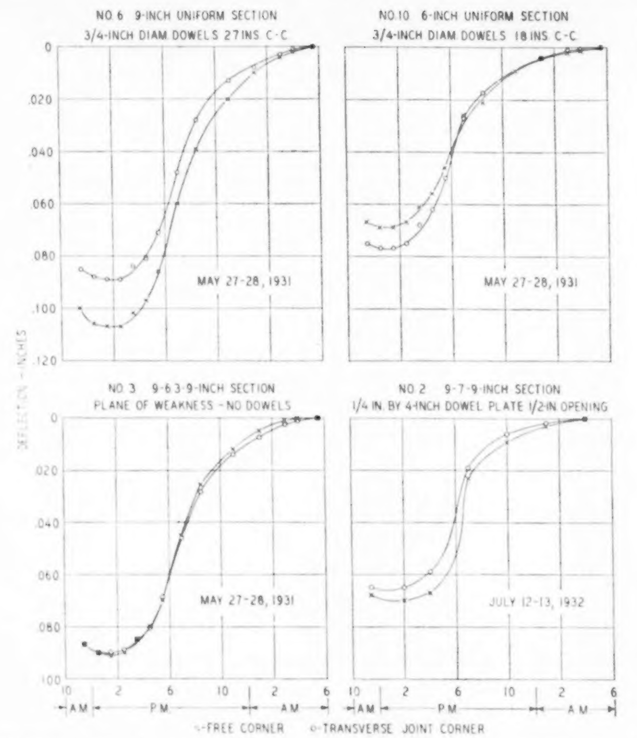

Figure 18.—Comparison of the Cycles of Temperature Warping Movements at the Transverse and Free Joint Corners of Typical Slabs.

plane-of-weakness joint (C–7) had this joint been cracked through at the time.

A study was made of the relative vertical deflections at the free and the joint edges of the various slabs to determine the magnitude of the restraint offered by the various transverse and longitudinal joints. The deflected or warped shapes of the slab axes were measured with a clinometer and the cycles of vertical displacement of slab corners were measured with micrometer dials. Typical deflection data obtained from these measurements are shown in figures 18 to 21, inclusive. Since there were no thermocouple installations in the majority of the slabs, it was not possible to determine exactly the time at which the different slabs were of constant temperature throughout their depth and hence were in the unwarped condition. Therefore, for these particular comparisons, the total change that occurred during a full daily cycle is used in each case.

The extent of the vertical displacements caused by temperature warping at both the free and the transverse joint corners of several of the test sections are shown in figure 18. Since the deflection resulting from warping is greater at the corner than at any other point along the edge, it seems reasonable that any restraining effect of the joint would be most apparent in deflection data obtained at the corners. For this reason any indications of restraint in this figure probably represent approximately maximum conditions.

Data for two doweled joints, a dummy joint, and a dowel plate are included in this figure. Data for the doweled joint with the dowels at 27-inch intervals indicate some resistance to warping, while those for the other doweled joints in which dowels are installed at 18-inch intervals actually show a greater deflection at the joint corner than at the free corner. This is

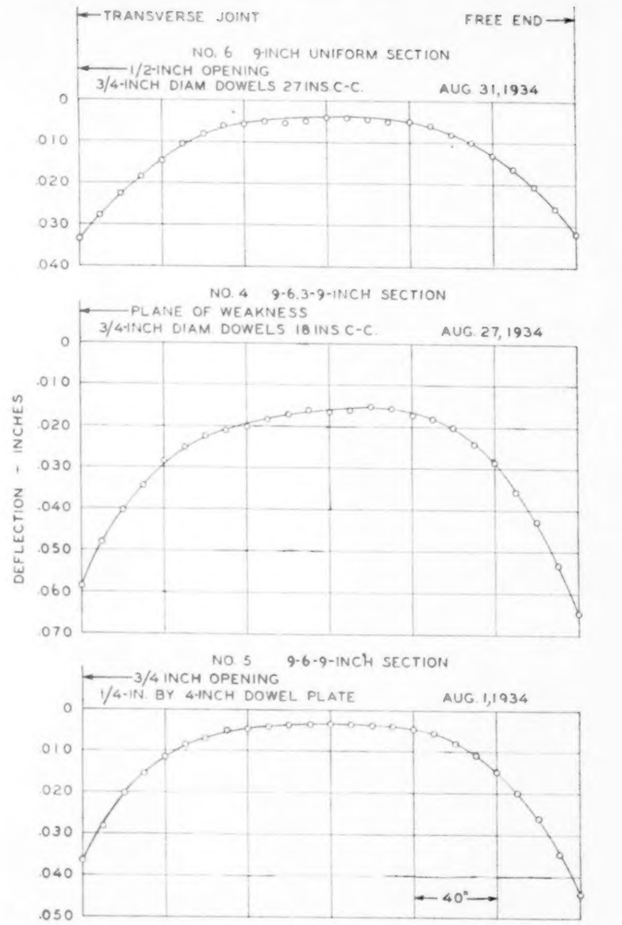

Figure 19.—Temperature Warping Deflections Along Free Edge of Typical Sections.

rather typical of much of the data obtained during these particular measurements. At times the free corner would show a slightly greater range of deflections than the joint corner and again, at some other time, the reverse would be true. The differences were generally so small as to be of no great importance in interpreting the data. The data for the transverse weakened-plane or dummy joint and for the dowel-plate joint indicate quite definitely that little or no restraint to warping is offered by either design.

In figure 19 the effects of temperature warping along the free edge of several of the sections are shown for days on which relatively large temperature variations occurred. The zero or base values for these curves were observations made at approximately 6 a. m. in each case, while the other set was made in the early afternoon at the time of maximum downward warping of the slab edges. An indication of the degree of restraint to warping offered by several transverse joints is obtained by comparing the warping at the transverse joint with that at the free end of the slab.

The joint types covered by figure 19 are the same as in the preceding figure, except that the dummy joint shown in figure 19 contains dowels and the dowel-plate joint has a different joint opening. The indications of this graph are in general agreement with those of figure 18 and, so far as these deflection data go, one would conclude that none of the designs of transverse joints included in these observations show any indi-

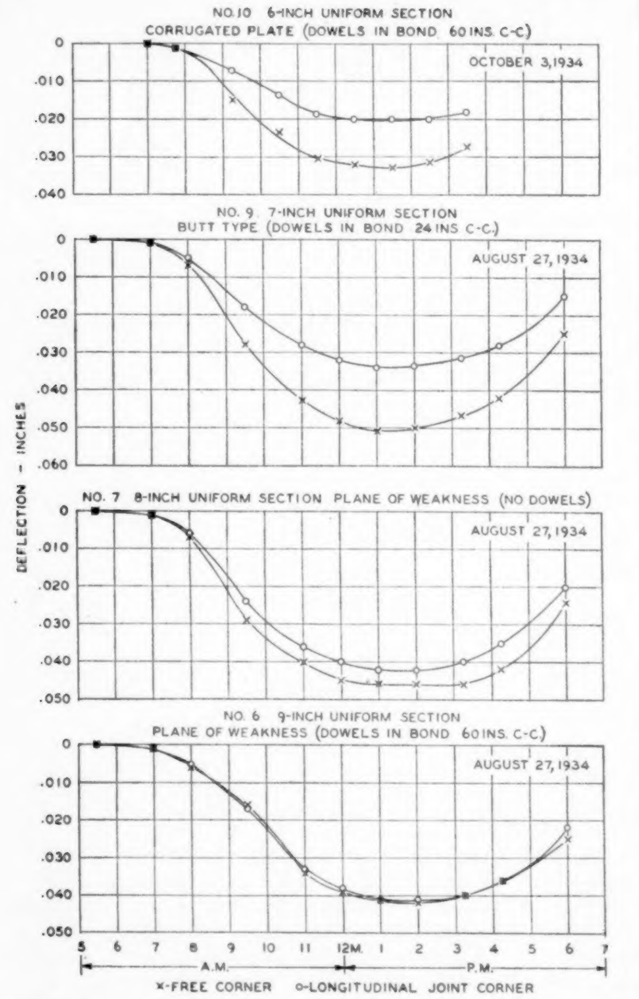

FIGURE 20.—COMPARISON OF THE CYCLES OF TEMPERATURE WARPING MOVEMENTS AT THE LONGITUDINAL AND FREE JOINT CORNERS OF THE SLABS OF UNIFORM THICKNESS.

cation of offering serious resistance to warping. Final judgment as to the efficiency of the various joints in permitting warping should be reserved until certain stress data to be presented later are examined.

CERTAIN TYPES OF LONGITUDINAL JOINT OFFERED NOTICEABLE RESTRAINT TO SLAB WARPING

In the case of the longitudinal joints the study of the effect of the joint design on the restraint to warping, based upon the deflection data, was restricted to the four sections having a constant slab thickness; in all of the others the thickening of the free edges prevented a direct comparison of deflection data obtained at the joint with those obtained at the free edge of the slab. Fortunately, the more important types of longitudinal joint are represented in these four sections, as shown in the following summary:

Types of longitudinal joint:	*Slab thickness* *Inches*
Corrugated plate with bars at 60-inch intervals	6
Butt joint (tarred felt) with bars at 24-inch intervals	7
Plane of weakness. No bars	8
Plane of weakness with bars at 24-inch intervals	9

Figure 20 shows the relative magnitudes of the vertical displacements measured at the free and longitudinal joint corners of each of these four sections over a full daily cycle. The method of measurement was the same as that described in connection with the discussion of transverse joint action.

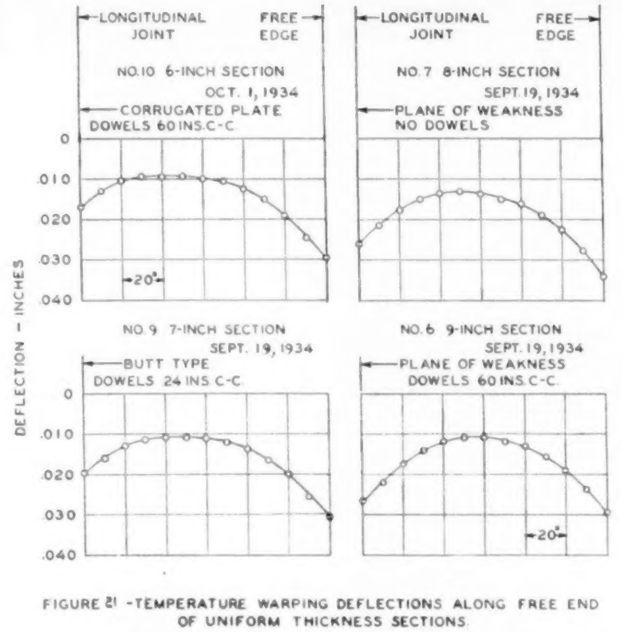

FIGURE 21.—TEMPERATURE WARPING DEFLECTIONS ALONG FREE END OF UNIFORM-THICKNESS SECTIONS.

These curves indicate that the weakened-plane type of joint, which provides a greatly reduced section at the joint, caused very little restraint to temperature warping even when crossed by bonded steel. The butt-type joint and the type that contains a deformed, metal separating plate both appear to offer noticeable resistance to warping action.

Restraint to warping in joints that are crossed by bonded steel results from a resisting moment which develops between the tension of the bonded steel bars and the compression in the abutting surfaces of concrete in the two slab edges as the joint closes when the slab begins to warp. The length of the moment arm depends upon the distance to which the effective section extends either above or below the plane containing the steel. With the position of this plane fixed and at some point typical of general practice as, for example, halfway between the two surfaces of the slab, a relatively deep groove in the upper surface such as is present in weakened-plane joints greatly reduces the moment arm for the condition of downward warping and effects a corresponding reduction in the restraint. Restraint to upward warping would not be relieved by such a groove, however. Joints that contain little or no reduction in section provide a greater moment arm and, with other conditions the same, will develop more restraint to warping.

Curves showing the change in slope and the extent of the warping that occurred across the free end of each of the constant-thickness slabs during some particular day are shown in figure 21. The data were obtained with the clinometer in the manner previously described. The evidences of restraint shown by these data are in accord with what was shown by figure 20.

Additional information on the relative restraint to warping of the various transverse joints was obtained as a result of stress determinations based on strain measurements made at the positions shown in quadrant

4 of figure 8. The two gages placed perpendicular to and 6 inches from the free edge were used as index gages and the average deformation measured at these points, when corrected by means of Poisson's ratio for the strain measured by the gage parallel to the slab edge, was used as a base for computing the strains caused by restrained warping at other points.[15]

Stress values obtained in this manner for typical cases of warping for certain of the transverse joints are given in table 4 and for the longitudinal joints in table 5. A comparison between the stresses found at corresponding points near the joint edge and free edge of a slab panel, for a given temperature condition, brings out the effect of the joint on the stresses caused by warping. For example, the stresses at a point 18 inches from the joint edge should be compared with those found at the same distance from the free edge.

TABLE 4.—*Warping stresses caused by the various transverse joints*

Date tested (1934)	Test section no.	Type of joint	Spacing of dowels	Longitudinal stress [1]: 6 inches from transverse joint	Longitudinal stress [1]: 18 inches from transverse joint	Longitudinal stress [1]: 18 inches from free end	Transverse stress [1]: 6 inches from transverse joint	Transverse stress [1]: 6 inches from free end
			Inches	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
July 19	7	Doweled	18	+5	+32	0	------	------
July 26	7	do	18	+20	+5	−50	------	------
June 28	4	Plane of weakness.	18	−68	−25	−35	−12	+59
June 29	4	do	18	−93	−35	−30	−14	+22
June 30	3	do	None	+69	+80	−10	−5	−65
July 3	3	do	do	+97	+100	+15	+42	−43
Sept. 20	2	Dowel plate	Continuous	−25	−22	+32	+56	+30
Sept. 28	2	do	do	−9	−42	+10	−14	−61
Sept. 10	4	Crack	None	−140	−140	−25	------	------

[1] A positive sign indicates tensile stress and a negative sign compressive stress in the upper surface of the slab.

TABLE 5.—*Warping stresses caused by the various longitudinal joints*

Date tested (1934)	Test section no.	Type of joint	Dowel spacing	Transverse stress [1]: 6 inches from joint	Transverse stress [1]: 18 inches from joint	Transverse stress [1]: 18 inches from free edge
			Inches	*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Lbs. per sq. in.*
July 7	[2] 3	Tongue	60	−53	−23	+40
July 11	3	do	60	−15	−10	+68
Aug. 22	[2] 5	do	60	−6	−57	−40
Oct. 3	10	do	60	−38	−60	−18
Sept. 18	9	Butt	24	+6	+28	+48
Sept. 19	9	do	24	−20	+8	+38
Sept. 26	[2] 2	do	48	−68	−80	−18
Oct. 2	2	do	48	−59	−90	−27
July 21	6	Plane of weakness	60	−41	−65	+2
July 24	6	do	60	−7	−42	+55
July 16	7	do	None	−20	−25	−15
July 17	7	do	None	−17	−32	+10

[1] A positive sign indicates tensile stress and a negative sign compressive stress in the upper surface of the slab.
[2] Thickened-edge slab.

DATA SHOW THAT JOINTS SHOULD OFFER A MINIMUM OF RESTRAINT TO WARPING

Since the deformations were all measured in the upper surface of the pavement for a condition in which the edges of the slab were warping downward, restraint to warping would be expected to cause an increase in the compressive stresses at the positions near the joint edge over those found at corresponding positions near the free edge.

A number of tests were made on thickened-edge slabs. The fact that a slab has thickened edges should not affect the stress comparisons so far as the transverse joints are concerned but the comparisons for the longitudinal joints are affected because the index gages, being at the thicker, outside edge of the slab, will record a larger deformation for a given degree of bending than will the gage at a corresponding position near the thinner, longitudinal joint edge. In the case of transverse stresses, the stresses themselves are small and the effect at the thickened edge does not appear to be important. In the case of the longitudinal stresses, however, the effect is more important and these data have been omitted from table 5 for this reason. It is indicated by both the deflection and stress data that none of the longitudinal joints causes any serious restraint to warping in the direction parallel to the joint.

It will be noted that the stress values given in tables 4 and 5 are generally small and that they tend to be erratic. It is believed that the tendency for the stress values to be erratic is caused in part by the variable behavior of the slabs, which was mentioned previously in connection with the deflection data, and in part by the small deformations and relatively few measurements involved. In most of the other stress determinations in these investigations it was possible to average a considerable number of observations of deformations which in themselves were of much greater magnitude than those being considered here. It may be said, however, that the stress data show no indications of serious restraint to warping in any of the designs tested.

Attention is called to the fact that the tests on the weakened-plane joints were made during the summer, when the joints were closed, a condition which should produce the maximum restraint to warping in slabs of the length used in these tests.

The crack in one of the slabs of section 4 was tested as a joint and, as shown by the data in table 4, appeared to exert a greater restraint to warping than any of the joint designs. There are probably two related causes for this. The first, and probably the most important, is the firm connection of the cracked panel with the adjoining uncracked panel by a tongue-and-groove type of longitudinal joint which exerted a stiffening effect on the cracked panel. The second is that the two broken edges of the cracked panel appeared to be tightly in contact at all times.

For warping to take place it is necessary to displace the two slabs abutting the crack longitudinally a slight amount. Except for some frictional resistance in the longitudinal tongue-and-groove joint the only force resisting this longitudinal movement is the resistance to deformation of the subgrade material, the magnitude of which varies with the length of the slab. In this particular case the resistance must have been quite small because the slab length was only 10 feet. In longer slabs, forces of considerable magnitude might easily be created at times when the concrete was in an expanded condition and the cracks tightly closed, although the presence of the crack tends to ameliorate the warping stress conditions in its vicinity.

The deflection and stress data just presented to show the relative degree of the restraint to warping offered by the various joints tested in this investigation are but a part of the data obtained although they are typical

[15] For a discussion of the formulas and methods of computing these stresses the reader is referred to the second report of this series published in PUBLIC ROADS, vol. 16, no. 9, November 1935.

in all instances. To some extent they are erratic but it is believed that in spite of this the deflection and stress data both indicate that none of the joint designs which were tested are sufficiently resistant to bending to offer serious restraint to warping. The data do point to the danger of designing joints which are resistant to bending because of the warping stresses that such designs are likely to cause to be developed at times when warping of the slab occurs. It is indicated that a fundamental structural requirement in joint designs should be that the resistance to bending in both directions, but particularly in a plane perpendicular to the direction of the joint, be a minimum in order that the stresses caused by warping restraint will be as small as possible.

STUDY MADE OF THE RELATIVE ABILITY OF VARIOUS JOINTS TO STRENGTHEN THE JOINT EDGE OF THE SLAB

The third group of tests of joints was planned to develop data that would show the relative effectiveness of the different designs from the standpoint of their ability to reduce the natural weakness of the slab at the joint edge. With the one exception of the thickened ends used at the transverse joint in section 1, this strengthening of the slab edge was accomplished through a transfer to the adjacent slab of a part of any load applied near the joint. The influence of the several designs on the deflections and on the stresses in the vicinity of the applied loads will be brought out in the discussion which follows.

Figures 22 to 25 contain data showing how the various parts of the different slabs deflect under the influence of loads applied at the points designated in the figures. The curves show the measured deflections for the two slabs abutting the joint under test, and, for comparison, the deflection of a corresponding edge or corner under the same load but lacking the support of a connecting joint. For example, in figure 22, which shows the deflections of the outside edge of the test section, a load applied on one side of the transverse joint deflected the two abutting slab ends in the manner shown by the crosses, while the same load placed near the free end of the outside edge produced the deflection of the free end which is indicated by the circles. Referring back to figure 8, which shows the location of the lines of clinometer points in the four quadrants, the crosses in figure 22 show deflections at points along the line A_3—A_2 for a load at C_3 while the circles show the deflections at points along the line E_3—A_3 for the same load at E_3.

In making an estimate of the ability of the various joints to reduce the deflection of the slab edge on which the load is applied, certain assumptions have been made:

1. If the joint design has a maximum of effectiveness in performing this function, a load placed on one side of but close to the joint edge should cause equal deflection of the two slab ends that abut the joint.

2. If the joint design is completely ineffective in this respect, a load on one side of, but close to, the joint should produce no deflection of the slab end on the opposite side of the joint.

The first and second assumptions serve as the basis for the first method of estimating joint effectiveness from the deflection data. The method will be described in a succeeding paragraph.

3. If the joint design has a maximum of effectiveness, the application of a given load at the joint should produce a deflection having a magnitude one-half as great as that which would be produced by the same load acting at an unsupported edge.

4. If the joint design is completely ineffective, a given load will cause the same deflection when applied at the joint edge and at a corresponding point of the free edge of the test slab.

The third and fourth assumptions are the basis of the second method of estimating joint effectiveness from the deflection data.

It is believed that all of the assumptions are correct for the condition of complete and uniform contact between the slab and the subgrade, because, for such a condition, the load-deflection relation is practically linear for loads within the safe stress range. It is indicated by these tests that this condition rarely prevails and that the extreme edges of the slab are in full contact with the subgrade only when they are warped downward.

Since the joint tests were made with the slabs in an unwarped condition brought about by the protective coverings described in the first report, the edges of the panels were not in full contact with the subgrade at the time the loads were applied, with the result that the load-deflection relation is not linear. A typical example of the character of this relation for a load applied at the free corner of one of the test sections is given in figure 26. It will be noted that for equal increments of load, each succeeding increment causes a somewhat smaller increment of deflection. This condition affects the third assumption because, for a given load, the free edges and corners are generally deflected more than the joint edges and corners. The third assumption therefore does not apply strictly to the conditions under which the tests were made and this fact should be considered in the application of these data.

On the basis of these four assumptions it is possible to estimate the extent to which the various joints are effective in reducing the deflection of the loaded joint edge, by making use of the deflection data given in figures 22 to 25, inclusive.

DEFLECTION OF LOADED SIDE OF JOINT ALWAYS EXCEEDED THAT OF ADJACENT SIDE

Two methods are available for making such an estimate. As noted previously, the first method makes use of the first and second assumptions, while the second method makes use of the third and fourth assumptions. Figure 27 shows the comparisons that are involved in each method of analysis.

While it is believed that the first method is probably the better measure of the ability of the joint design to reduce deflection because it does not involve the third assumption, both methods are of some value and will be used in the comparisons which follow.

Figure 22 shows the deflections along the outside edge of the various slabs caused by a load at points E_3 and C_3 and gives some indication of the effectiveness of the joint constructions in reducing the deflections caused by loads applied at the joint corners. For a load at point E_3 the shape of the deflected slab is shown between points E_3 and A_3, while for a load at point C_3 the shape is shown between points A_3 and A_2.

Figure 23 shows the extent of the deflections along the centerline of a 10- by 20-foot panel caused by loads applied either at point I_3 or G_3 and indicates the effect of the various transverse joints on the deflection caused by a load acting at a transverse joint at a point away from a longitudinal edge. For a load at point I_3 the shape of the deflected centerline is shown between

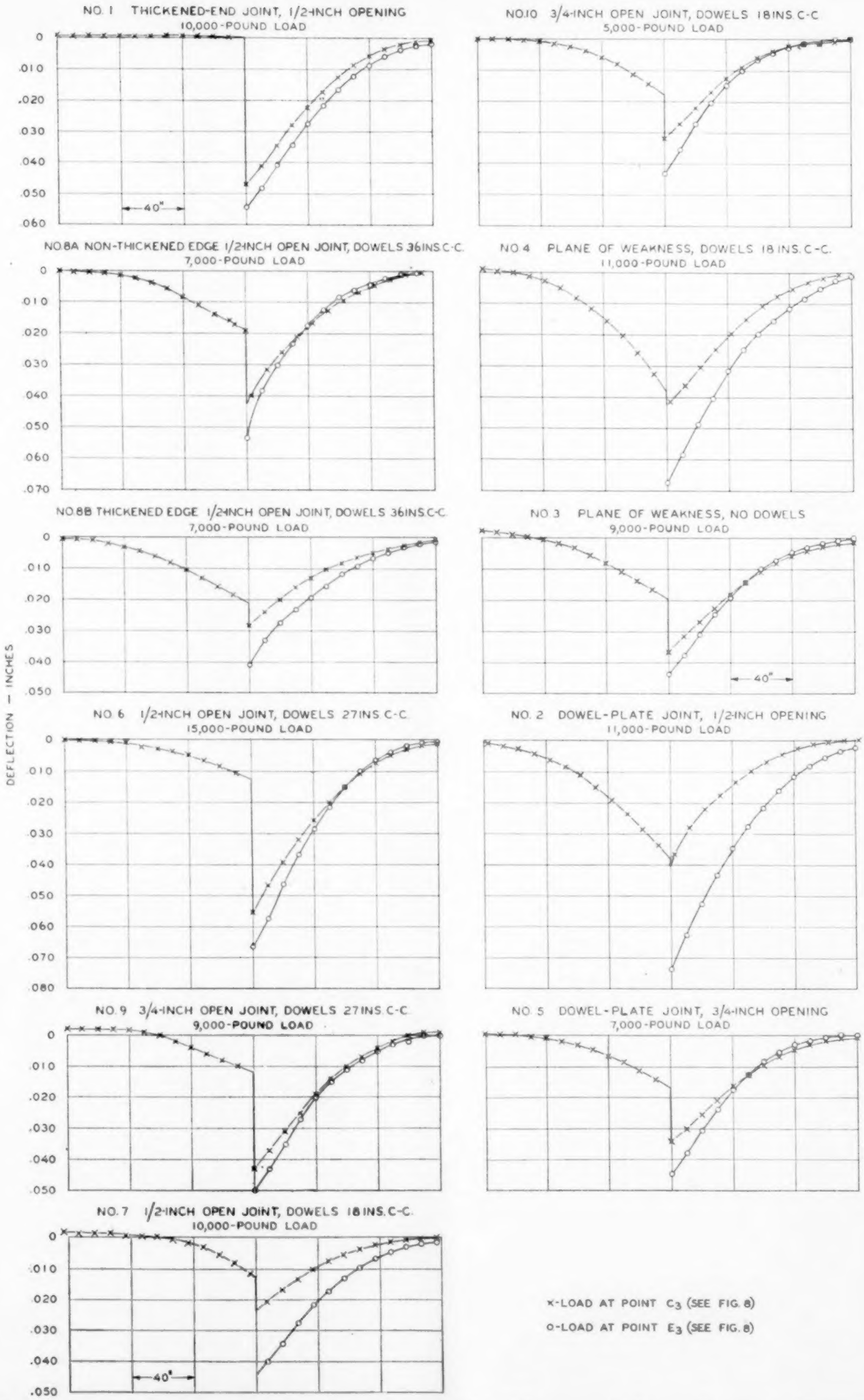

FIGURE 22.—COMPARISON OF DEFLECTIONS ALONG FREE EDGE OF TEST PANELS FOR LOADS PLACED AT FREE CORNER AND AT CORRESPONDING TRANSVERSE JOINT CORNER. ALL DOWELS USED IN TRANSVERSE JOINTS WERE ¾ INCH IN DIAMETER AND 36 INCHES LONG.

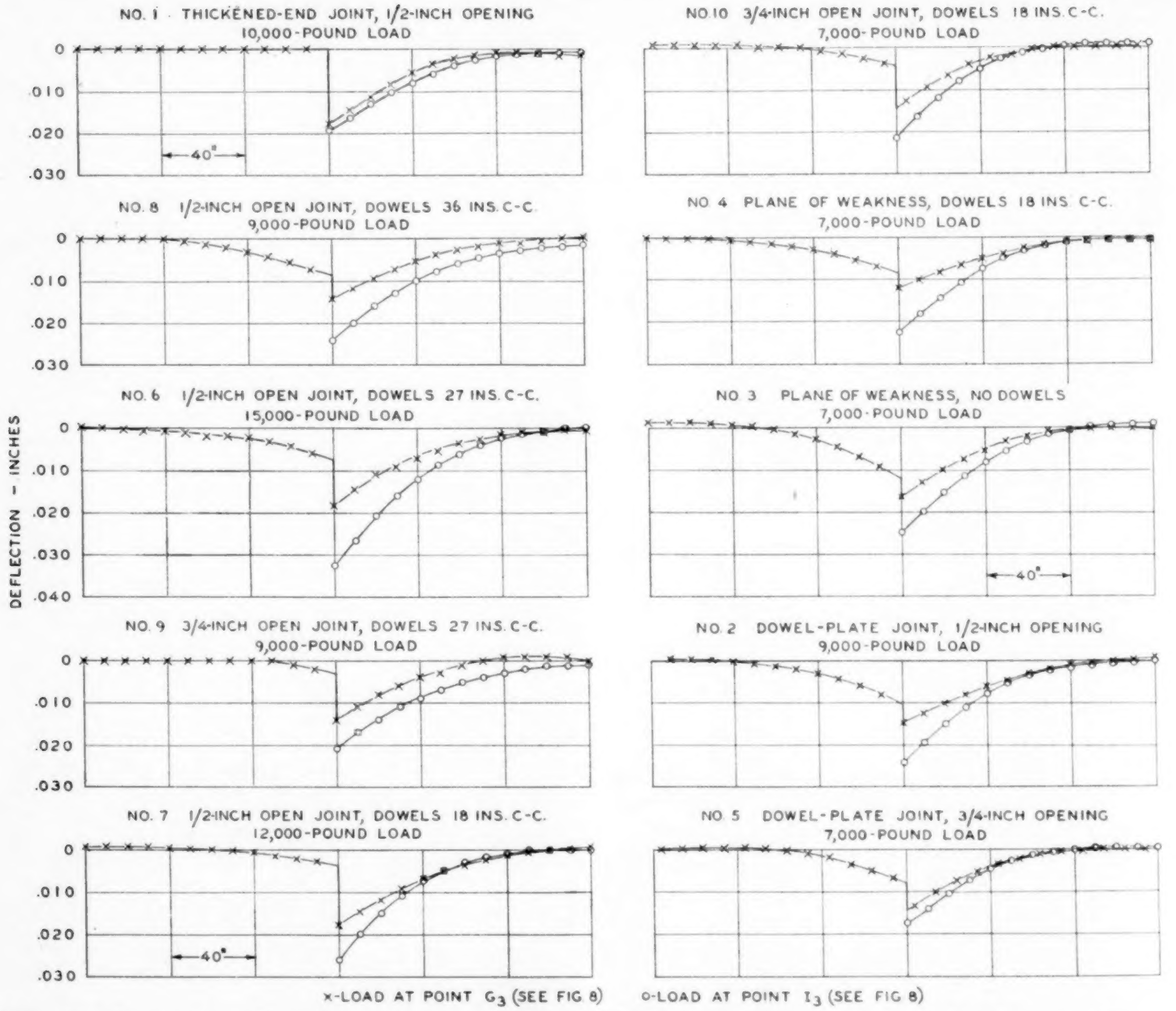

FIGURE 23.—COMPARISON OF DEFLECTIONS ALONG LONGITUDINAL CENTERLINE OF TEST PANELS FOR LOADS PLACED AT MIDPOINT OF FREE END AND AT CORRESPONDING POINT AT TRANSVERSE JOINT. ALL DOWELS USED IN TRANSVERSE JOINTS WERE ¾ INCH IN DIAMETER AND 36 INCHES LONG.

points I_3 and H_3 while for a load at G_3 the deflections are shown between points H_3 and H_2.

It will be observed that the data for transverse joint 1–1 (thickened slab ends) are included in these figures. Since there is a complete separation of slab ends in this construction and no load transfer is intended, these particular load-deflection data do not have the same significance as do those that apply to the other sections. As would be expected from the design, the two ends of the 10- by 20-foot panel behave in identical fashion and a load applied on one side of the transverse joint causes no deflection of the slab end on the other side of the joint.

Figure 24 shows the deflections along the end of the slabs caused by loads applied either at points E_3 or F_3 and indicates the effectiveness of the various longitudinal joints in reducing the deflections caused by loads applied at the joint corners of the slab. The shape of the slab is shown along the line E_3—F_3 for a load applied at E_3 and, similarly, with a load at F_3, the deflections between the points E_3 and I_4 are shown.

Figure 25 shows the deflections along the transverse centerline of a slab panel caused by loads applied at points A_3 and B_3, and these data indicate the ability of the various longitudinal joints to reduce deflection for loads applied near the longitudinal joint and at some distance from a transverse joint. For a load acting at point A_3 the deflections are shown between points A_3 and B_3, while for a load acting at B_3 the deflections are shown between points A_3 and H_4.

In the case of the thickened-edge slabs it is not possible to compare directly the deflections at the free and the longitudinal joint edges of the slabs. For this reason the comparisons in figures 24 and 25 are restricted to the constant-thickness sections.

In the data just presented for the transverse joints, it will be observed that the loaded side of the joint always deflects more than the adjacent side to which load is transmitted by the joint structure. The difference is greater for the joints containing the round dowel bars than for those which contain the dowel plates. There is also a variation in the magnitude of this differ-

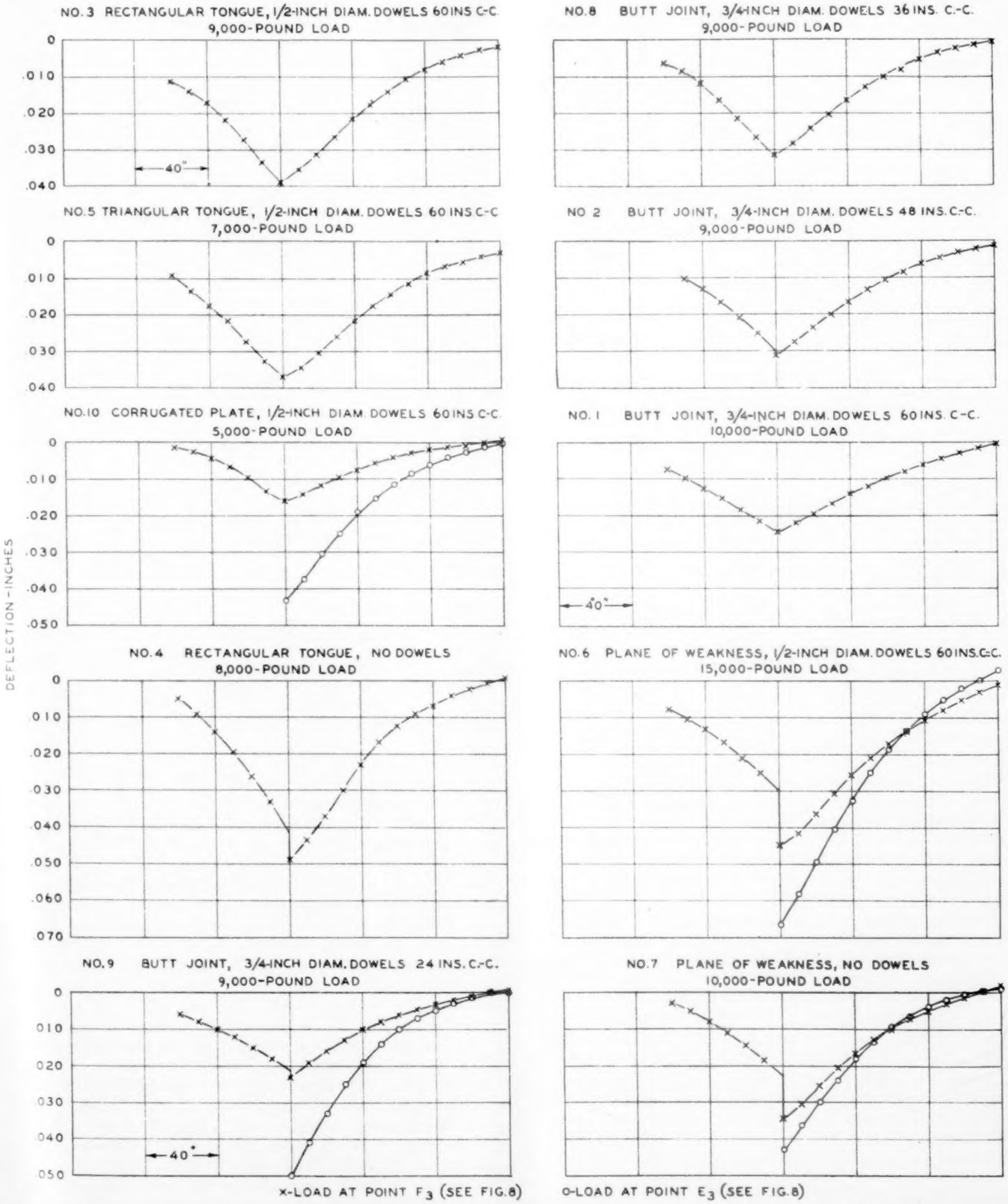

Figure 24.—Comparison of Deflections Along Free End of Test Panels for Loads Placed at Free Corner and at Corresponding Longitudinal Joint Corner. Dowels ¾ Inch in Diameter Used in Joints C–1, C–2, C–8, and C–9; Dowels ½ Inch in Diameter Used in Joints C–3, C–5, C–6, and C–10. All Dowels in Bond.

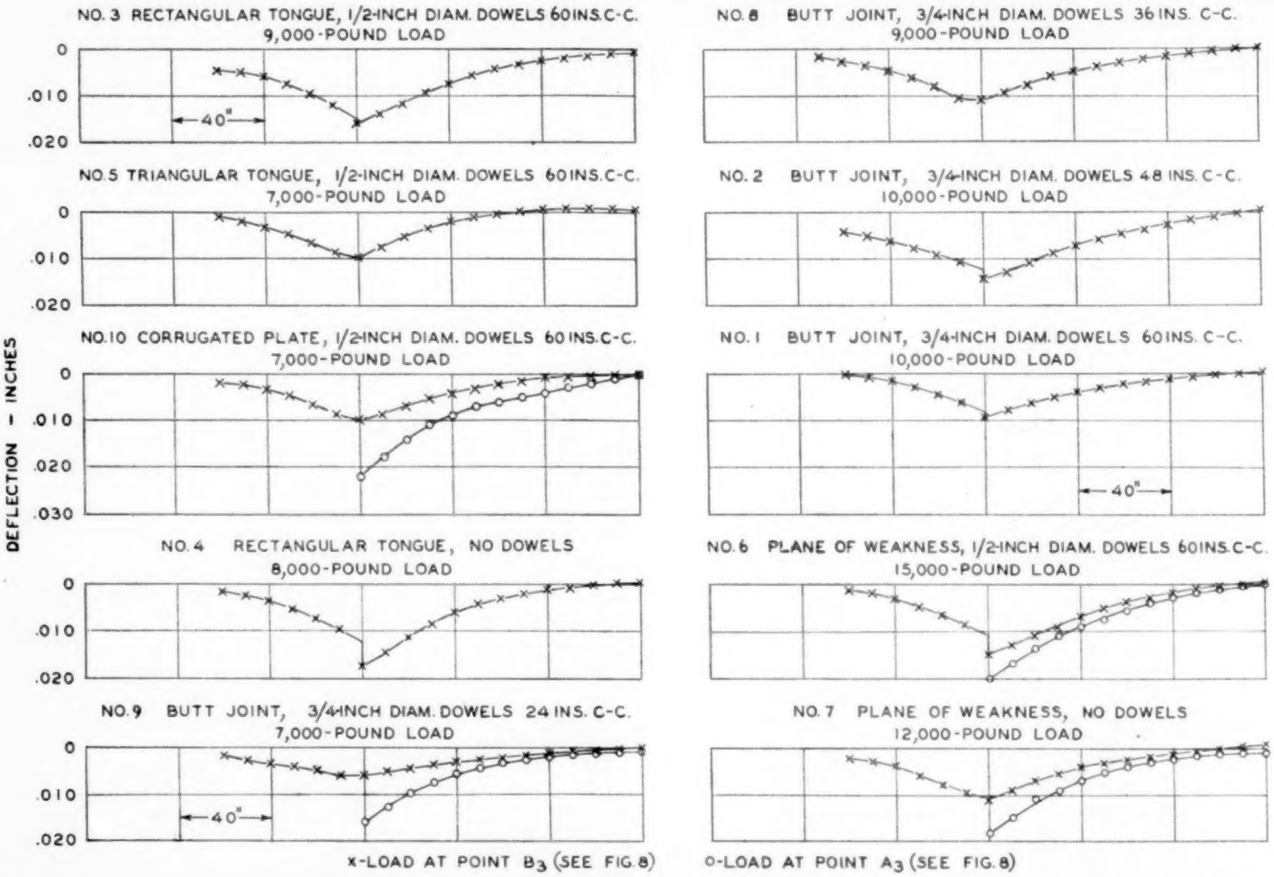

FIGURE 25.—COMPARISON OF DEFLECTIONS ALONG TRANSVERSE CENTERLINE OF TEST PANELS FOR LOADS PLACED AT MIDPOINT OF FREE EDGE AND AT THE CORRESPONDING POINT AT LONGITUDINAL JOINT. DOWELS ¾ INCH IN DIAMETER USED IN JOINTS C-1, C-2, C-8, AND C-9. DOWELS ½ INCH IN DIAMETER USED IN JOINTS C-3, C-5, C-6, AND C-10. ALL DOWELS IN BOND.

ence among the joints that contain the dowel bars. This variation might possibly result from a lack of stiffness in the dowel bars or from a looseness of the dowels in the concrete, or from a combination of the two. In order to develop information regarding the relative importance of each of these factors, a series of loads was applied on each of the four quadrants of two of the sections at a joint corner (point C) and at several points directly over dowel bars at some distance from the corners (near point G).

For the purpose of these comparisons it is assumed that, for a load applied on one side of a doweled joint, so long as the dowels are firmly in bearing on each side of the joint, the deflection rate of the adjacent slab end will bear a constant relation to the deflection rate of the loaded slab end. The ratio of the one to the other will be a constant as each load increment is applied. When the loaded edge begins to deflect, the adjacent edge will not follow immediately if any deficiency in dowel seating is present, and this lag will be evident as a variation in the ratio between the load deflection rates for those increments of load applied while the seating deficiency exists.

DATA ON LOAD-DEFLECTION MEASUREMENTS AT JOINT EDGES PRESENTED

Figure 28–A shows the load-deflection relation for both the loaded and unloaded joint ends at the corner of the panel, for sections 6 and 7, for a series of uniform increments of applied load. From the corresponding deflection increments "k" and "p", the mean slope ratio, $\frac{p}{k}$, was calculated for each increment of load and these values are plotted as ordinates to the curves shown in figure 28–B.

This graph indicates that the dowels at the corner of section 6 were slightly loose, because the ratio is smaller for the first load increments than for the later ones. Apparently the 6,000-pound load was sufficient to take up the dowel looseness completely, and it is noted that this load caused a maximum deflection of the loaded corner of about 0.016 inch. In the case of section 7 there is no indication of dowel looseness, the value of the ratio being practically constant for all load increments.

The flexibility of the dowels is indicated by the fact that for the upper increments of load the deflection of the adjacent slab corner is but 50 or 60 percent of that of the corner on which the load was applied.

The deflections of the loaded corners of the two sections are approximately the same for a given load despite the difference in slab thickness. The amount of support received from the adjacent slab corner is not the same in the two cases, however, and this probably accounts to a large extent for the effect noted.

Figure 29 contains data obtained in similar tests on section 7 in which the loads were applied directly over the dowels but at some distance from the longitudinal edges of the slab (near point G). These are average curves from tests made at several points on the same

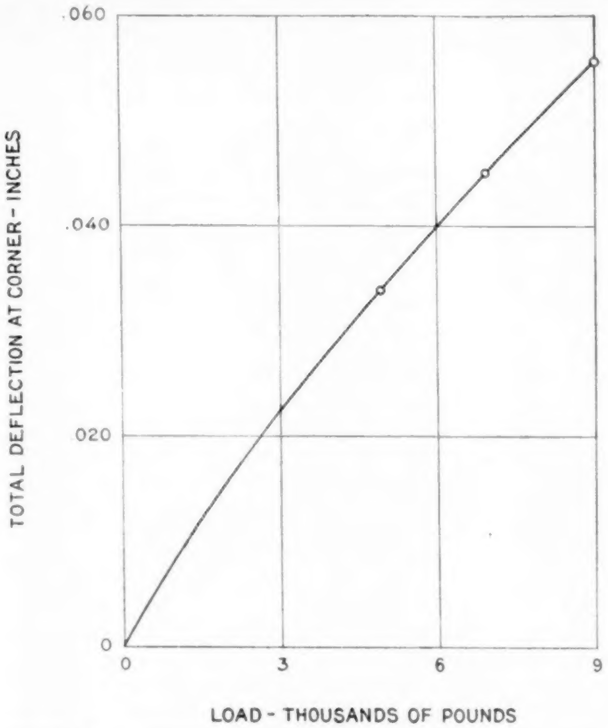

FIGURE 26.—TYPICAL LOAD-DEFLECTION RELATION FOR A LOAD ACTING AT A FREE CORNER.

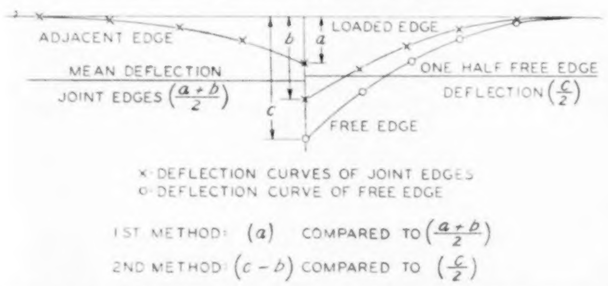

FIGURE 27.—METHODS OF ESTIMATING THE EFFECTIVENESS OF JOINTS FOR REDUCING SLAB DEFLECTION ON THE BASIS OF LOAD-DEFLECTION DATA.

slab and are plotted in the same manner as the data shown in the preceding figure. In this case it appears that the differences between the deflections of the two slab ends should be attributed entirely to dowel flexure, there being no indication of dowel looseness.

A comparison of the values of the slope ratios shown in the last two figures shows that the value is smaller for loads applied near the joint at points remote from the corner than for loads applied near the slab corner. This indicates that to obtain the same percentage of deflection across a doweled joint at all points a stiffer transfer medium is required at points that are away from the slab corner than is required in the immediate vicinity of the corner. Also a stiffer medium is required for thick slabs than for thin slabs.

It is thought that the data shown in the last two figures furnish an explanation for most of the differences found in the deflection data presented in figures 22 to 25, inclusive. In general, it was found that only a very few of the dowels were sufficiently loose to produce any apparent detrimental effect on the ability of the joint to transfer load. This is rather surprising when consideration is given to the very small deflections involved in this action. During construction considerable care was taken to place the concrete around the dowels properly, but typical field methods were employed in all of the operations and the condition of these dowels is considered representative of what can be obtained where dowels are carefully installed in first-class construction.

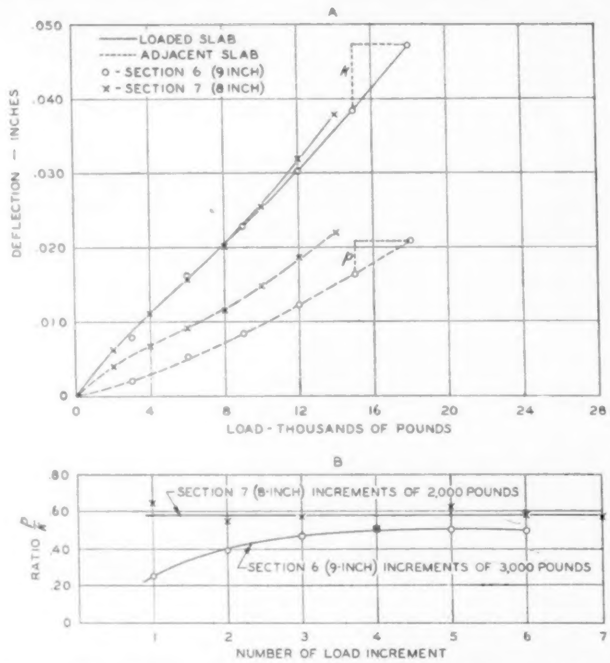

FIGURE 28.—DATA ON STUDY OF DOWEL STIFFNESS AND DOWEL LOOSENESS, TRANSVERSE JOINT CORNERS.

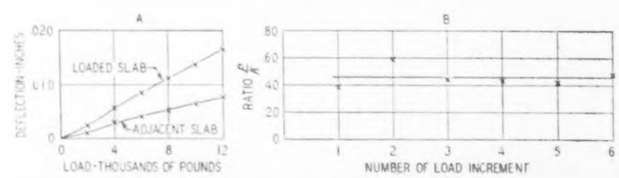

FIGURE 29.—DATA ON STUDY OF DOWEL STIFFNESS AND DOWEL LOOSENESS, TRANSVERSE JOINT EDGES (SECTION 7).

Somewhat similar studies of the deflections of concrete pavement slabs caused by loads acting near joints have been reported by the State highway organizations of Georgia [16] and Michigan.[17] The data given in these two reports are in general agreement with those which have been presented in figures 22 to 25, inclusive.

TWO METHODS USED TO MEASURE EFFECTIVENESS OF JOINTS IN REDUCING DEFLECTION OF THE LOADED JOINT EDGE

Tables 6 and 7 were developed from the data shown in figures 22 to 25, inclusive, for the purpose of bringing out more directly the comparisons that appear to be significant. Table 6 applies to the transverse joints and table 7 to the longitudinal joints.

In columns 5 and 8 of these tables a comparison is made between the sum of the maximum deflections of the two slab edges abutting the joint in question and the deflection of a corresponding point at a free edge of the same slab, the load being the same in both cases. Referring to figure 27, the comparison of deflections

[16] Tests of Load Transmission across Joints in Concrete Pavements, by Searcy B. Slack, Engineering News-Record, vol. 107, no. 2, July 9, 1931, p. 53.
[17] Tests of Aggregate Interlock at Joints and Cracks, by A. C. Benkelman, Engineering News-Record, vol. 111, no. 8, Aug. 24, 1933, pp. 227-232.

involved in the computation of the values shown in columns 5 and 8 is expressed as a percentage of the free edge deflection by the formula

$$\frac{(a+b)-c}{c}$$

Columns 6 and 9 show the effectiveness of the various joints in reducing the deflection of the loaded edge by means of a comparison of the deflection of the unloaded joint edge with the mean deflection at the joint. This is the first method previously mentioned and the values are computed by the formula

$$\frac{a}{\frac{(a+b)}{2}}$$

Columns 7 and 10 give similar comparative values developed by the second method, the reduction in deflection effected by the joint being compared to one-half of the deflection of the free edge (a reduction of one-half of the free edge deflection would mean perfect joint action). This relation is expressed by the formula

$$\frac{c-b}{\left(\frac{c}{2}\right)}$$

TABLE 6.—*Various relations between the deflections caused by loads placed at the free and the transverse joint edges of slabs*

Test section no.	Type of joint	Spacing of dowels	Joint opening	Tests at slab corners: Sum of deflections at joint corner exceeds that at free corner by [1]	Tests at slab corners: Joint effectiveness estimated by first method	Tests at slab corners: Joint effectiveness estimated by second method	Tests at midpoint of slab end: Sum of deflections at midpoint of joint end exceeds that at midpoint of free end by [1]	Tests at midpoint of slab end: Joint effectiveness estimated by first method	Tests at midpoint of slab end: Joint effectiveness estimated by second method
1	2	3	4	5	6	7	8	9	10
		Inches	*Inches*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
8	Doweled	36	½	18	73	50	−5	76	83
6	do	27	½	3	37	33	−22	57	88
9	do	27	¾	10	44	28	−19	35	67
7	do	18	½	−18	70	94	−19	33	65
10	do	18	¾	14	71	53	−14	47	67
4	Plane of weakness	18		19	95	74	−10	84	96
3	do	None		28	70	32	14	84	68
2	Dowel plate		½	5	97	92	4	82	77
5	do		¾	13	66	48	27	70	34

[1] A negative sign indicates that the sum of the deflections of the two edges abutting the joint under test is smaller than the deflection of the corresponding point at the free edge of the slab.

The values in table 6 indicate that for a given load the sum of the deflections at the corner of a transverse joint is nearly always greater than the deflection of the corresponding free joint corner. At the transverse joint edge at a distance from the corner, in some cases the sum of the deflections at the joint edge exceeds the corresponding deflections at the free edge and in other cases it does not.

Earlier in the discussion it was shown that the load-deflection relation at a slab corner is not linear because the slab is not completely in contact with the subgrade when the deflection begins and the conditions of support change gradually as the deflection progresses. It was shown in figure 26 that for equal increments of load the resulting increments of deflection gradually

TABLE 7.—*Various relations between the deflections caused by loads placed at the free and longitudinal joint edges of the slabs*

Test section no.	Type of joint	Type of tongue	Spacing of dowels [1]	Tests at slab corners: Sum of deflections at joint corner exceeds that at free corner by [2]	Tests at slab corners: Joint effectiveness estimated by first method	Tests at slab corners: Joint effectiveness estimated by second method	Tests at midpoint of slab end: Sum of deflections at midpoint of joint end exceeds that at midpoint of free end by [2]	Tests at midpoint of slab end: Joint effectiveness estimated by first method	Tests at midpoint of slab end: Joint effectiveness estimated by second method
1	2	3	4	5	6	7	8	9	10
			Inches	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
3	Tongue	Rectangle	60		100			97	
5	do	Triangle	60		100			100	
10	do	Corrugated	60	−26	100	126	−12	100	111
4	do	Rectangle	None		91			83	
9	Butt		24	−12	96	108	−24	100	124
8	do		36		100			100	
2	do		48		98			92	
1	do		60		100			94	
6	Plane of weakness.		60	12	80	65	28	85	52
7	do		None	33	79	40	21	100	80

[1] All dowels across longitudinal joint were fully bonded.
[2] A negative sign indicates that the sum of the deflections of the two edges abutting the joint under test is smaller than the deflection of the corresponding point at the free edge of the slab.

decrease as the magnitude of the deflection increases. It is thought that this is the reason that the combined deflections at a transverse joint corner nearly always exceed the corresponding deflection at a free corner. The free corner, on account of its relatively greater deflection, is able to offer more resistance to deflection. The values given in columns 5 and 8 of these tables are significant because they illustrate the point just discussed and help to explain differences in the values of joint effectiveness as estimated by the two methods of analysis.

When the loads are applied at the slab edges at some distance from a corner, the deflections are all small, relatively. The effect of varying subgrade support is therefore much less.

It is apparent from table 6 that the two methods of analyzing the deflection data to determine the relative effectiveness of the transverse joint designs in reducing the deflection of the loaded edge of a joint give different values for the same joint design. For the corner loading the first method of calculation, with one exception, yields higher values, while for the interior edge condition the relation between the values obtained by the two methods of comparison is quite irregular. The most probable cause for this irregularity has already been mentioned. The values obtained by the first method probably give the better idea of the ability of the joints to transfer load, while those obtained by the second probably give a better indication of the relative ability of the joints to reduce the maximum deflection of the loaded edge. Neither method gives a complete measure of the structural efficiency of the joint to which it is applied, however.

WEAKENED-PLANE JOINTS CONTAINING DOWELS WERE EFFECTIVE IN REDUCING SLAB DEFLECTION

From table 6 it appears that the transverse joints that depend upon round dowels for their connection are not as effective in reducing maximum deflections as might reasonably be expected. The data for the

joints having dowels spaced 18 and 27 inches apart, respectively, indicate that the effectiveness of the joint in reducing corner deflection is increased as the dowel spacing is decreased. The joint containing the dowel bars with a 36-inch spacing is not in line with the others in respect to this relation. Why this should be is not known, although the fact that this particular transverse joint was installed in the section having the lip-curb cross section may have been at least partly responsible. The two halves of this section (longitudinally) were different, so that there was opportunity for but one-half as many comparisons as on the other sections. Also, because of the presence of the lip curb on the upper surface, certain difficulties in duplicating loading conditions were always present in the tests on this section.

So far as the values computed from the deflection data indicate, dowel spacing within the limits of the tests did not influence the effectiveness of the doweled joints in reducing the deflection of the loaded edge. Neither do they show any consistent difference attributable to the differences in the width of the joint opening used in these tests.

The tests on the transverse plane-of-weakness or dummy joints were made in November when the slabs were contracted and the joints were opened slightly. The comparative values for the weakened-plane joints containing dowels (particularly those computed by the first method) indicate a very effective construction so far as ability to reduce slab deflection is concerned. The joint without the dowels is also indicated as being quite effective by these values. It should be remembered in this connection that the slab length in which the joints were used is but 20 feet.

It is indicated further by the values in table 6 that the two joints containing the one-fourth by 4-inch dowel plates are effective in reducing slab deflection. When used with a joint opening of one-half inch the effectiveness of a dowel plate of this thickness seems to be noticeably greater than when used with a three-fourths-inch joint opening.

Table 7 contains similar comparative values for the longitudinal joints computed by both methods wherever possible. One is struck immediately by the generally higher order of values in this table when compared to those given in the preceding table for the transverse joints. This is a direct reflection of the better structural connection obtained in the longitudinal joints where little or no change in joint width occurs and no provision for slab expansion was necessary.

It will be noted in table 7 that the values computed for the corners of the longitudinal joints in sections 9 and 10 show that the combined deflections at the joint are less than the deflection at the corresponding free corners. This relation is the reverse of that generally shown in table 6 for the transverse joints.

As stated earlier, it was possible to make comparisons between the deflections of free and longitudinal joint edges of slabs only for sections of constant thickness and even then only two of the four constant-thickness sections (sections 9 and 10) were suitable for the desired comparison. The other two constant-thickness sections were of the plane-of-weakness type and only one of these contained bonded steel across the joint. It is believed, however, that the relations indicated for sections 9 and 10 will probably be found in any constant-thickness section in which the dummy-joint construction is not used and the slab edges are

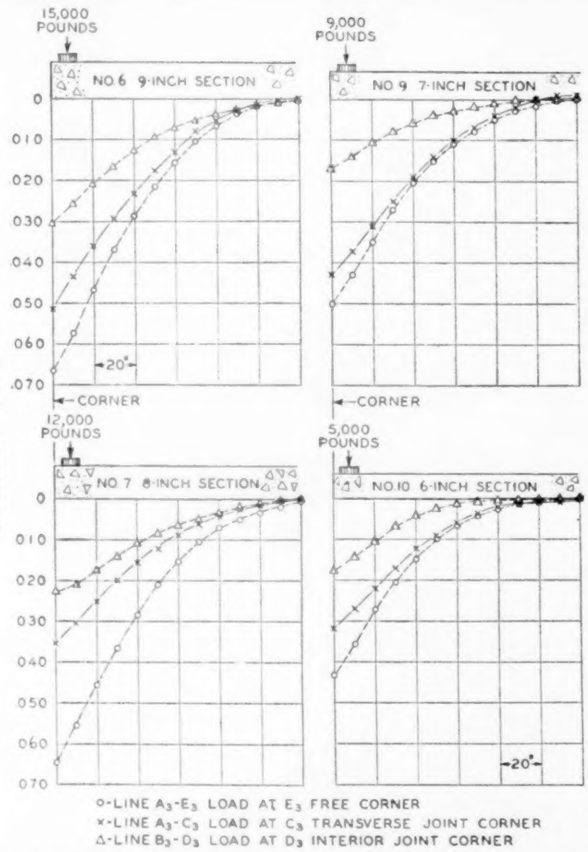

FIGURE 30.—COMPARISON OF DEFLECTIONS OF SLAB EDGES AT THE FREE TRANSVERSE JOINT AND INTERIOR JOINT CORNERS FOR THE UNIFORM-THICKNESS SECTIONS.

held in contact by bonded steel members across the joint. That such a design permits the development of a resisting moment during deflection has already been pointed out. The effect of this moment is to stiffen the joint against deflection under load and this would produce the relation indicated by the values in table 7. The magnitude of the moment that it is possible to develop will depend upon the effective depth of the slab at the joint, the amount and position of the steel capable of taking tension, any opening of the joint, and other factors.

Table 7 shows that the sum of the deflections at the longitudinal joint exceeds that at a corresponding point at the free edge in the two plane-of-weakness joints. For the other two sections of constant thickness (sections 9 and 10), the sum of the deflections at the joint is less than that at the free edge because of the effect of resisting moments due to the design of the joints. This has been discussed earlier in this report. The effect of the deep groove in the plane-of-weakness joint is apparent if the data for section 6 are compared with those for section 10.

Figure 30 shows a comparison of the deflections of the different corners of the four constant-thickness sections under a given load. The greatest deflection occurs at a completely free or unattached corner. Some reduction in deflection is brought about when the corner is attached on one side by a transverse joint. A still greater reduction is effected when the corner is supported along the other side by a longitudinal joint capable of transferring load. The joints and combinations of joints are different in each of the four sections.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS
1936 AND 1937 FUNDS
AS OF AUGUST 31, 1936

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 5,208,287				$ 78,130	$ 39,065		$ 466,812	$ 233,406	23.2	$ 4,935,816
Arizona	3,564,709	$ 1,669,709	$ 1,320,304	97.8	489,034	413,183	4.3	175,441	133,165	9.2	1,698,057
Arkansas	4,275,929										4,275,929
California	9,508,671	1,184,591	676,027	37.1	7,560,895	4,335,202	158.6	4,402,977	2,538,563	136.0	1,958,879
Colorado	4,575,144	1,838,205	1,028,873	81.6	2,415,286	1,348,812	76.5	299,605	167,779	10.0	2,029,680
Connecticut	1,582,913				1,087,965	541,780	15.0				1,041,133
Delaware	1,218,750	181,626	90,813	30.3	258,123	129,061	4.1	165,121	82,561	7.7	916,315
Florida	3,315,558	674,678	337,339	23.8	418,048	209,024	12.5	360,093	180,047	10.5	2,589,149
Georgia	6,336,443	223,572	111,786	21.3	1,986,829	962,890	136.2	1,133,409	566,704	56.5	4,695,062
Idaho	3,065,304	683,815	403,280	101.4	2,317,029	1,386,715	168.7	259,551	155,315	19.2	1,119,994
Illinois	10,325,922	1,521,778	760,654	30.5	7,922,269	3,921,780	119.0	2,229,283	1,114,641	40.9	4,528,847
Indiana	6,184,258	2,126,566	1,062,010	74.8	3,925,540	1,962,697	130.6	1,936,480	967,704	63.3	2,191,847
Iowa	6,466,628	2,804,859	1,325,969	247.0	4,131,030	1,955,894	189.4	1,711,268	824,266	83.6	2,360,499
Kansas	6,631,085	1,248,231	623,489	326.9	4,549,805	2,248,455	479.7	1,597,003	798,451	186.2	2,960,690
Kentucky	4,611,955	1,894,397	928,263	128.3	308,333	154,166	14.6	1,230,444	580,941	36.1	2,948,585
Louisiana	3,557,930	957,603	478,802	35.8	1,615,244	807,622	55.4	559,718	279,859	15.3	1,991,648
Maine	2,177,197	1,465,495	731,950	42.8	436,392	218,196	15.9				1,227,051
Maryland	2,050,870				181,230	90,615	3.3	854,879	382,807	10.2	1,577,447
Massachusetts	3,485,364				607,382	303,691	5.1	1,497,728	748,864	10.9	2,432,809
Michigan	7,668,768	2,175,235	1,086,121	87.4	10,169,669	5,082,109	324.2	2,315,650	1,157,825	64.7	342,713
Minnesota	6,849,307	4,594,132	2,114,058	305.9	3,450,991	1,637,247	197.4	1,228,568	614,284	65.8	2,483,718
Mississippi	4,387,636							883,327	441,664	62.2	3,945,972
Missouri	7,601,200	2,360,662	1,177,908	322.1	4,545,043	2,272,521	142.1	3,957,001	2,028,390	282.9	2,122,381
Montana	5,122,333	2,520,163	1,410,907	318.5	1,839,019	1,029,666	100.2	1,107,200	581,730	68.1	2,100,030
Nebraska	5,167,930	1,195,001	597,421	79.8	3,199,641	1,619,183	293.8	1,144,487	558,243	79.6	2,393,083
Nevada	3,189,479	911,306	785,716	153.5	1,030,714	892,289	123.6	129,584	111,275	6.0	1,400,199
New Hampshire	1,218,750	606,373	296,960	19.2	155,703	77,691	3.3	274,045	136,737	3.9	707,362
New Jersey	3,352,469	149,200	74,580	1.1	2,950,263	1,388,601	39.2	805,284	402,642	4.6	1,486,646
New Mexico	3,990,023	1,478,670	898,654	132.1	1,841,118	1,141,839	85.9	740,438	450,334	98.5	1,499,196
New York	12,306,710	602,000	301,000	14.4	12,359,529	6,096,452	208.2	4,821,300	2,399,500	101.1	3,509,758
North Carolina	5,879,466	939,398	469,615	132.9	3,227,109	1,612,159	363.6	282,890	122,648	22.1	3,675,044
North Dakota	3,918,269				191,740	101,206	.1				3,817,063
Ohio	9,131,204	239,940	119,970	5.1	6,009,803	2,968,018	83.2	1,570,772	696,538	8.5	5,346,679
Oklahoma	5,884,927	1,501,063	788,543	57.9	1,631,631	855,167	49.9	1,487,508	781,685	55.6	3,459,532
Oregon	4,089,711	901,250	548,914	31.6	2,798,939	1,621,286	96.6	1,056,471	636,216	34.5	1,283,294
Pennsylvania	10,695,448	1,104,837	552,150	19.9	7,286,311	3,642,120	106.1	3,785,215	1,888,503	49.3	4,612,676
Rhode Island	1,218,750							322,636	161,318	4.8	1,057,432
South Carolina	3,381,337				263,175	120,968	17.4	3,469,273	1,574,750	291.7	1,685,619
South Dakota	4,078,647	628,130	344,404	54.4	776,701	455,082	126.3	32,074	17,586	12.4	3,261,575
Tennessee	5,268,270	1,040,902	519,879	43.3	1,140,268	570,134	43.0	568,099	284,049	24.6	3,894,207
Texas	15,548,821	7,657,787	3,823,655	409.2	6,567,531	3,278,165	365.5	619,455	296,195	5.3	8,150,806
Utah	2,826,960	1,177,497	843,262	89.7	775,547	552,287	43.1	155,383	111,843	7.8	1,319,568
Vermont	1,218,750	753,320	375,994	42.3	769,310	375,311	25.0	372,410	165,898	8.4	301,547
Virginia	4,559,200	697,873	346,582	23.8	2,148,852	1,074,426	86.5	1,884,523	942,261	82.0	2,195,931
Washington	3,904,738	1,097,103	576,992	34.9	3,247,198	1,708,649	135.4	887,556	466,802	38.7	1,152,295
West Virginia	2,716,754	168,656	84,328	7.2	1,206,059	602,518	45.5	294,899	147,449	4.6	1,882,459
Wisconsin	6,090,504	1,104,132	551,489	41.4	5,276,527	2,524,772	197.3	1,438,119	681,611	59.1	2,332,632
Wyoming	3,121,972	2,274,424	1,390,181	250.8	1,085,990	653,047	182.9	514,629	316,042	61.8	762,702
District of Columbia											
Hawaii	1,218,750				467,856	231,167	8.2				987,583
TOTALS	243,750,000	56,354,179	29,958,842	3957.8	126,700,800	65,212,928	5,082.4	55,028,608	27,929,091	2,327.4	120,649,139

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF AUGUST 31, 1936

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$ 4,151,115				$ 3,449,243	$ 3,449,243	111.9	$ 512,692	$ 512,692	17.2	$ 189,180
Arizona	2,569,841	$ 1,312,602	$ 1,243,230	31.2	1,669,785	1,217,937	99.5	41,845	19,394	6.1	89,280
Arkansas	3,352,061	1,071,508	1,069,211	80.3	2,048,610	2,031,963	242.8	170,139	169,389	25.5	81,498
California	7,747,928	1,984,574	1,892,616	101.9	5,740,581	5,610,824	145.6	160,841	160,841	1.5	83,647
Colorado	3,395,263	947,101	946,294	64.2	1,128,097	1,128,097	34.0	62,474	62,474	6.0	1,258,398
Connecticut	1,418,709	4,268	3,995		407,166	385,452	2.2	267,235	267,135	3.8	762,128
Delaware	900,310	107,945	107,945	23.3	380,787	380,787	23.8	174,293	145,453	13.6	266,125
Florida	2,597,144	100,691	100,691	8.3	2,144,394	2,126,521	78.2	248,707	248,707	7.3	121,225
Georgia	4,988,967	143,732	143,732	6.1	714,058	714,058	51.6	261,116	261,116	14.1	3,870,062
Idaho	2,222,747	707,401	704,408	64.7	1,523,755	1,486,892	118.5				31,447
Illinois	8,694,009	1,859,451	1,859,119	101.9	6,294,211	6,294,211	338.7	510,557	510,557	26.9	30,121
Indiana	4,941,255	355,983	355,983	9.0	4,653,198	4,232,181	217.9	106,585	106,585	7.8	246,505
Iowa	4,991,664	913,951	865,919	174.0	3,232,114	3,121,849	233.1	1,049,724	987,479	104.2	16,418
Kansas	4,994,975	671,298	671,293	104.9	4,258,188	4,248,377	268.0	75,305	75,305	1.6	
Kentucky	3,726,271	619,600	619,600	111.4	1,848,765	1,848,765	182.8	703,549	703,549	42.0	554,357
Louisiana	2,890,429				2,111,801	1,833,230	118.4	1,011,940	908,770	56.2	148,429
Maine	1,676,799	491,346	491,081	21.1	962,886	961,991	42.3	167,663	167,663	7.4	56,063
Maryland	1,750,738	60,038	60,038	3.0	309,355	309,355	11.9	1,093,042	929,702	26.7	451,643
Massachusetts	3,262,885				934,611	934,611	4.6	508,641	508,641	3.3	1,819,633
Michigan	6,301,414	2,187,200	2,187,200	98.8	3,946,221	3,886,191	185.5	53,000	53,000	3.2	175,023
Minnesota	5,277,145	2,069,500	1,819,484	377.0	3,192,585	2,690,222	448.6	890,267	633,253	80.0	134,186
Mississippi	3,457,552	346,841	346,133	28.2	2,599,971	2,596,545	167.8	156,756	156,756	11.6	358,119
Missouri	6,012,652	2,132,562	2,125,800	546.7	3,248,653	3,197,723	222.6	624,498	621,490	21.0	67,639
Montana	3,676,416	1,931,027	1,931,027	105.3	1,677,010	1,677,010	80.0				68,379
Nebraska	3,870,739	745,501	731,696	97.7	2,445,139	2,413,151	239.6	265,317	265,317	25.8	460,575
Nevada	2,243,074	1,211,443	1,178,417	49.6	787,623	781,080	49.2				283,577
New Hampshire	945,225	181,048	179,415	7.3	474,432	451,535	18.2	119,425	92,522	5.6	221,752
New Jersey	3,129,805				2,158,920	2,158,920	18.4	265,805	251,986	4.6	718,899
New Mexico	2,871,397	1,264,733	1,264,733	109.6	1,048,747	1,048,747	67.9	147,248	147,248	7.2	410,669
New York	11,046,377	1,710,223	1,682,798	16.6	9,171,795	8,787,419	149.5	251,200	251,200	1.8	324,959
North Carolina	4,720,173	510,944	510,944	25.8	3,257,981	3,225,392	232.3	309,740	276,900	15.8	706,937
North Dakota	2,867,245	248,767	248,767	49.7	1,739,288	1,737,001	183.1	606,714	606,714	62.6	274,763
Ohio	7,670,815	508,380	508,380	8.2	4,067,273	4,006,433	106.5	2,136,900	2,026,437	147.6	1,129,564
Oklahoma	4,580,670	608,678	607,067	58.5	2,349,804	2,347,709	216.6	881,471	880,950	102.4	744,944
Oregon	3,038,642	904,981	900,706	39.5	1,850,431	1,761,940	101.1	539,972	261,372	26.2	114,624
Pennsylvania	9,347,797	533,858	508,095	21.0	1,675,411	1,635,478	64.4	1,453,206	1,453,206	71.2	5,751,018
Rhode Island	989,208	232,376	232,376	5.5	747,159	747,159	13.3				9,673
South Carolina	2,702,012	293,927	288,583	34.4	1,767,901	1,699,562	161.0	228,511	228,003	21.1	485,863
South Dakota	2,976,454	753,058	753,058	194.1	1,391,871	1,391,871	208.4	498,803	498,803	60.0	332,723
Tennessee	4,192,460	551,529	551,529	22.2	2,061,306	2,061,306	83.1	679,839	679,839	30.0	899,786
Texas	11,989,350	5,989,725	5,426,804	550.2	6,461,749	5,926,115	545.4	711,913	585,923	22.3	50,508
Utah	2,067,154	695,307	657,917	77.4	1,137,362	1,001,838	86.8	248,525	241,609	11.5	165,790
Vermont	924,306	322,046	304,381	9.8	697,956	576,130	12.0	8,000	8,000		35,795
Virginia	3,652,667	1,398,014	1,356,131	509.0	1,630,684	1,600,408	458.4	477,113	473,528	72.3	222,600
Washington	3,026,161	1,279,513	1,154,226	95.1	1,834,682	1,631,529	64.0	280,130	239,789	5.5	617
West Virginia	2,231,412				1,543,384	1,539,846	59.9	484,209	483,477	20.4	208,090
Wisconsin	4,823,884	1,277,781	1,132,526	115.4	4,030,986	3,549,104	213.7	192,202	136,604	11.6	5,651
Wyoming	2,219,155	501,569	501,563	32.9	1,596,569	1,596,554	106.1				121,038
District of Columbia	949,496	825,733	801,703	7.5	143,498	143,498	.9				4,295
Hawaii	926,033	95,216	82,604	1.3	549,947	541,611	7.6				301,818
TOTALS	195,000,000	42,662,969	41,109,218	4,259.6	115,097,943	110,725,371	6,897.7	19,637,112	18,299,378	1,212.5	24,866,033

CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF AUGUST 31, 1936

STATE	APPORTIONMENT	COMPLETED					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS
		Estimated Total Cost	Works Program Funds	NUMBER			Estimated Total Cost	Works Program Funds	NUMBER			Estimated Total Cost	Works Program Funds	NUMBER			
				Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,034,617	$ 94,744	$ 94,744	2			$ 3,202,207	$ 3,202,207	33		1	$ 597,427	$ 597,427	6		10	$ 140,239
Arizona	1,256,099	398,678	394,444	5			650,897	639,452	6			86,220	63,832	1			158,371
Arkansas	3,574,060	635,497	633,117	17	1		1,767,605	1,763,489	25	4		1,035,487	1,033,353	14		29	144,101
California	7,486,362	639,111	626,616	7	4		6,775,110	6,543,321	45	3							316,426
Colorado	2,631,567	684,147	663,147	12			839,851	839,851	11			41,994	41,994		1		1,086,575
Connecticut	1,712,684						68,067	68,067		1		578,069	505,991	3	1		1,138,626
Delaware	418,239						143,486	120,000	1								298,239
Florida	2,827,883	284,120	284,120	2	2		1,562,718	1,560,407	16	3		309,257	309,257	9			674,099
Georgia	4,895,949						304,545	304,545	6	1		309,863	309,863	10			4,281,541
Idaho	1,674,479	361,899	361,899	7			754,934	754,934	10	1		8,732	8,732			4	548,914
Illinois	10,307,184	266,677	266,677	8			7,035,438	7,035,438	46	3		788,738	788,738	7			2,216,331
Indiana	5,111,096	4,202	4,202	1			4,866,887	4,743,768	36	11		413,314	89,585			161	273,541
Iowa	5,600,679	253,060	241,000	7	2		3,597,738	3,521,397	71	7		1,238,847	1,233,797	19	1	6	604,485
Kansas	5,246,258	219,579	219,579	3			4,892,292	4,892,292	48	1		134,387	134,387	2		3	
Kentucky	3,672,387	15,290	15,290	1			2,506,758	2,217,026	20			195,636	195,636	2	3		1,244,435
Louisiana	3,213,467						814,208	814,208	7	1		1,594,628	1,550,163	14	3		849,097
Maine	1,426,861	176,568	176,235	5			595,651	595,060	11			406,267	406,267	3		2	249,300
Maryland	2,061,751						408,723	408,723	3	2		1,070,839	1,040,332	4	2	17	612,696
Massachusetts	4,210,833	10,069	10,069	1			1,662,731	1,662,731	12	2		695,455	695,455	6			1,842,577
Michigan	6,765,197	816,125	816,125	10	3		5,332,927	5,288,427	32	4							660,645
Minnesota	5,395,441	555,434	555,434	23		9	3,413,077	3,407,027	53	9	22	610,990	610,990	7		10	821,990
Mississippi	3,241,475	25,677	25,677	2	1		2,048,953	2,048,953	42	2		383,843	383,843	4		1	783,002
Missouri	6,142,153	45,994	45,994	1			5,473,454	5,406,570	39			788,008	665,674	6		2	23,915
Montana	2,722,327	1,164,330	1,164,330	17	4		1,429,652	1,429,652	19	2							128,345
Nebraska	3,556,441	418,265	418,265	13			1,748,130	1,748,130	56	1	5	1,270,689	1,270,689	10		18	119,356
Nevada	887,260	306,096	306,096	7			397,409	397,409	3			205,556	183,755	1		5	
New Hampshire	822,484	34,094	34,094	1			417,861	417,861	2	4	1	12,337	12,337				358,192
New Jersey	3,983,826						1,278,281	1,278,281	7	2		433,620	433,620	5			2,271,925
New Mexico	1,725,286	370,259	370,259	8			629,110	629,110	4	1		575,874	575,874	5			150,043
New York	13,577,189	46,040	46,040		2		9,843,984	9,583,138	32	30		771,355	771,355	1	5		3,176,656
North Carolina	4,823,958	113,110	113,110	1	3		2,171,205	2,171,205	30	7		856,779	840,380	6	4	61	1,699,263
North Dakota	3,207,473	64,305	64,305	2			1,941,898	1,940,898	38			42,034	42,034	1	1		1,160,236
Ohio	8,439,897						2,441,311	2,294,346	16	1		2,572,373	2,485,784	15	4		3,659,767
Oklahoma	5,004,711	806,150	806,150	17	1		1,449,661	1,449,661	20	1		1,202,957	1,202,957	17	2		1,545,943
Oregon	2,334,204	126,698	126,698	1	2		2,258,196	2,133,484	15	4		74,022	74,022	1		2	
Pennsylvania	11,483,613	152,737	121,370	12			4,558,765	4,241,594	35	10		2,256,893	1,919,528	11	5		5,201,120
Rhode Island	699,691	236,879	236,879		1		418,195	417,129	4								45,683
South Carolina	3,059,956	154,754	151,450	3		1	1,210,424	1,202,722	24	5	2	580,239	579,829	11		32	1,125,955
South Dakota	3,249,086	188,494	188,494	10			1,033,679	1,033,679	23			722,742	722,742	14	1	19	1,304,171
Tennessee	3,903,979	28,389	28,389	1		2	527,940	527,940	11	1		1,479,064	1,479,064	18	1	18	1,868,586
Texas	10,855,982	699,096	699,096	18	2		7,463,176	7,456,802	88	11		2,435,354	2,382,315	18	2	150	317,769
Utah	1,230,763	31,147	29,757		1		608,107	608,107	9			305,778	265,000	3			327,899
Vermont	729,857	164,678	164,678	3	3		297,333	295,975	4	2		283,298	221,830	3	1	11	47,375
Virginia	3,774,287	172,632	172,632	11			1,577,240	1,465,216	25	2		828,230	828,230	18	4	11	1,308,209
Washington	3,095,041	276,868	272,068	5	2		2,066,935	2,066,903	11	7	2	510,695	510,695	6	2	5	245,375
West Virginia	2,677,937						426,806	426,806	4	1		908,973	908,973	16	1	4	1,342,157
Wisconsin	5,022,683	310,180	310,180	4			3,789,631	3,789,631	28	6		318,308	286,040	4			636,832
Wyoming	1,360,841	55,366	55,365	2			736,216	736,211	6			205,606	205,605	2			363,660
Dist. of Columbia	410,804						425,564	396,804	3								14,000
Hawaii	453,703						296,218	295,333	3			226,162	158,370	2			
TOTALS	196,000,000	11,407,438	11,314,074	250	34	12	110,161,184	108,271,920	1093	153	33	30,366,939	29,026,344	305	45	581	47,387,662

U. S. GOVERNMENT PRINTING OFFICE: 1936

PUBLICATIONS *of the* BUREAU OF PUBLIC ROADS

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Report of the Chief of the Bureau of Public Roads, 1929. 10 cents.

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Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

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An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

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SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

CURRENT STATUS OF UNITED STATES PUBLIC WORKS ROAD CONSTRUCTION

AS PROVIDED BY SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT (1934 FUNDS) AND BY THE ACT OF JUNE 18, 1934 (1935 FUNDS)

AS OF AUGUST 31, 1936

STATE	APPORTIONMENTS		COMPLETED				UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR NEW PROJECTS	
	Sec. 204 of the Act of June 16, 1933 (1934 Fund)	Act of June 18, 1934 (1935 Fund)	Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	Estimated Total Cost	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds	Mileage	1934 Public Works Funds	1935 Public Works Funds
Alabama	$ 8,370,133	$ 4,259,842	$ 14,760,438	$ 8,203,670	$ 3,123,411	739.4	$ 882,512	$ 160,296	$ 722,216	35.9		$ 368,935	2.1	$ 6,167	$ 45,280
Arizona	5,211,960	2,641,935	8,989,767	5,204,513	2,579,241	543.0	12,500		12,500					7,447	50,194
Arkansas	6,748,335	3,428,049	10,908,494	6,635,881	3,303,793	618.4	89,816		89,606	1.5	$ 58,020	18,148	2.8	54,434	16,502
California	15,607,354	7,932,206	30,601,146	15,589,249	7,754,205	760.9	115,025		115,016			820		18,106	62,165
Colorado	6,874,530	3,486,006	11,209,862	6,831,431	3,441,857	639.8								43,099	44,149
Connecticut	2,865,740	1,454,868	4,302,962	2,758,269	1,130,201	71.1	247,903	36,318	182,743	2.9				71,154	141,924
Delaware	1,819,088	923,395	2,676,358	1,818,804	815,188	128.3	53,319		53,319	.5		52,205	1.1	284	2,682
Florida	5,231,834	2,661,343	8,735,667	5,175,534	2,193,368	299.6	389,901		358,854	9.3		15,955	.8	56,300	93,165
Georgia	10,091,185	5,113,491	12,359,292	9,128,990	2,563,026	700.5	1,944,071	599,924	1,278,095	125.2	84,072	77,718	2.0	278,199	1,194,652
Idaho	4,486,249	2,277,486	6,622,657	4,416,568	1,787,381	496.8	429,790		425,402	4.7	42,905	37,043		26,776	27,659
Illinois	17,570,770	8,921,401	24,327,271	16,392,511	7,076,173	665.8	2,906,297	1,075,510	1,724,140	50.0	3,837	27,164	1.0	98,911	93,924
Indiana	10,037,843	5,088,963	14,710,560	9,775,380	4,241,443	450.3	925,856	206,489	690,066	38.0		119,945	.3	55,974	37,509
Iowa	10,055,660	5,118,361	15,361,180	10,055,660	4,708,561	1,218.5	433,921		409,800	8.8					
Kansas	10,089,604	5,117,675	15,349,642	10,003,063	5,021,408	1,131.1	226,121	77,553	60,318	14.4				8,988	35,948
Kentucky	7,517,359	3,818,311	11,584,128	7,448,139	3,303,863	798.1	466,110	59,003	389,509	14.7		9,116	.1	10,217	115,823
Louisiana	5,828,591	2,963,932	8,971,463	5,755,322	2,591,893	249.9	174,054	56,550	117,504	9.5		76,300	6.5	16,720	178,235
Maine	3,369,917	1,711,586	5,222,823	3,337,541	1,657,506	193.1	17,807	10,573	7,234					21,803	46,846
Maryland	3,564,527	1,810,058	4,986,774	3,241,542	751,108	130.5	712,090	240,565	426,560	26.0		380,742	4.9	82,420	251,648
Massachusetts	6,597,100	3,350,474	9,437,847	6,552,734	2,239,899	113.2	846,878		846,878	2.4		91,944	.1	44,366	171,754
Michigan	12,736,227	6,452,568	19,989,169	12,696,114	5,696,115	737.3	660,225		620,798	30.9	40,113		7.8		135,655
Minnesota	10,656,569	5,425,551	16,093,373	10,498,684	4,781,947	1,633.2	427,260	12,666	368,822	16.8	60,099	15,765	6.6	85,120	259,017
Mississippi	6,978,675	3,540,227	12,242,596	6,673,799	2,645,958	704.4	1,081,680	242,417	804,151	39.6	4,437	34,895	.9	58,022	55,223
Missouri	12,180,306	6,173,740	17,090,128	11,408,448	4,629,416	1,433.3	2,364,500	683,247	1,426,150	18.0		108,538	.4	88,611	9,636
Montana	7,439,748	3,769,734	11,571,712	7,324,443	3,574,147	1,033.6	267,308	80,883	186,425	24.7	18,440		.1	15,982	9,163
Nebraska	7,828,961	3,964,364	12,804,172	7,780,128	3,614,698	1,018.4	298,898		254,394	25.0		79,440	8.7	48,633	15,832
Nevada	4,545,917	2,302,356	7,065,806	4,545,917	2,253,348	760.4	25,004		25,004			22,580			1,425
New Hampshire	1,909,839	969,462	2,969,119	1,904,951	931,597	77.7	6,482		6,482					4,888	31,384
New Jersey	6,346,039	3,220,879	7,893,655	6,060,580	1,330,707	81.1	1,949,631		1,509,378	9.7	189,353	175,000	5.1	96,106	205,794
New Mexico	5,792,935	2,941,700	8,750,019	5,743,010	2,800,392	762.8	50,695		38,402	2.3	40,093		3.6	9,831	102,906
New York	22,330,101	11,327,921	38,331,568	21,548,282	9,935,219	807.1	2,200,970	678,700	1,149,599	16.1	50,047	186,135	.7	53,072	56,967
North Carolina	9,522,293	4,840,941	14,523,813	9,029,510	4,269,879	1,329.1	504,733	239,457	265,276	21.4	223,558	200,873	11.1	29,767	104,913
North Dakota	5,804,448	2,938,967	7,913,502	5,599,400	1,620,493	2,064.0	751,739	40,701	658,029	66.2	25,017	169,075	17.5	139,331	491,369
Ohio	15,484,592	7,865,012	23,829,944	15,313,142	6,718,114	770.9	1,099,390	112,165	958,450	24.1		52,706	1.4	59,285	135,742
Oklahoma	9,216,798	4,685,180	14,447,839	9,147,192	4,237,720	804.4	239,170	67,007	172,163	1.8		7,995	.4	2,599	267,302
Oregon	6,106,896	3,097,814	9,795,192	5,976,067	2,916,729	465.4	216,524	64,950	84,161	3.3		30,007		65,879	66,917
Pennsylvania	18,891,004	9,590,788	28,102,637	18,144,308	8,597,505	1,015.6	966,751	448,017	425,140	42.3	31,460	276,282	2.5	267,219	291,861
Rhode Island	1,998,708	1,014,572	3,140,657	1,998,708	1,008,873	89.1						3,806			1,893
South Carolina	5,459,165	2,770,954	7,381,230	5,234,172	1,894,523	590.6	765,168	97,255	645,143	34.5	11,408	59,684	2.1	116,329	171,604
South Dakota	6,011,479	3,047,643	8,844,678	5,775,621	2,493,829	1,515.0	482,823	38,883	443,940	33.1	8,598	42,500	25.0	188,377	67,375
Tennessee	8,492,619	4,302,991	13,099,274	8,492,619	3,585,905	480.2	553,239		553,239	12.5		113,346	9.5		50,500
Texas	24,244,024	12,291,253	37,402,430	23,895,669	11,630,525	2,775.1	872,353	284,494	552,648	10.9		500	7.1	63,861	107,580
Utah	4,194,708	2,132,691	7,191,769	4,160,916	1,991,960	590.6	148,028		108,741	.3	32,292	31,990		1,500	
Vermont	1,867,573	948,007	3,046,595	1,863,531	840,488	137.9	109,523	3,922	90,893	2.4		11,291	.7	121	5,335
Virginia	7,416,757	3,765,387	11,512,618	7,281,756	3,283,852	590.2	206,492	17,720	187,616	27.7	50,432	152,837	12.5	66,848	141,083
Washington	6,115,867	3,106,412	9,366,032	6,098,252	3,022,950	302.7					13,790	59,632	.3	3,825	23,830
West Virginia	4,474,234	2,280,335	6,043,846	4,319,738	1,458,208	205.6	796,912	84,032	656,373	16.9		66,425	.7	70,464	99,329
Wisconsin	9,724,881	4,941,837	15,373,330	9,683,993	4,851,910	619.7	27,006		27,006			37,916		40,888	25,006
Wyoming	4,501,327	2,287,712	6,873,312	4,466,218	2,195,823	1,037.1	16,616	11,729		.7		21,300		23,380	70,589
District of Columbia	1,918,469	973,842	2,740,809	1,909,584	831,090	20.7	120,199		84,699	1.5				8,885	58,053
Hawaii	1,871,062	949,778	2,597,023	1,870,383	169,089	50.6	740,023		736,462	5.2		23,434		679	20,793
TOTALS	394,000,000	200,000,000	609,145,578	384,769,936	170,096,534	34,452.1	28,823,313	5,731,026	20,949,344	831.7	987,971	3,259,987	146.4	2,511,067	5,694,135